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Stormwater Management Planning and Design Manual

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Stormwater Management Planning and Design Manual

March 2003

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PREFACE

The "state-of-the-art" of stormwater management has been rapidly evolving and this manual is one step in this evolutionary process. The manual provides technical and procedural guidance for the planning, design, and review of stormwater management practices. It is important that the manual be viewed as a tool for understanding the performance requirements of stormwater management projects and not as a rulebook for all stormwater management solutions.

The manual provides practical guidance which has been found effective in specific circumstances. However, users must exercise judgement and flexibly adapt the guidance provided. Stormwater management solutions need to consider specific site conditions and this must be recognized when applying the guidance provided in the manual.

It is not the intent of the Ministry to limit innovation with the manual. Significant effort has been made to write the manual in a manner that does not inadvertently restrict creative solutions. The Ministry encourages the development of innovative designs and technologies. Where the designer can show that alternate approaches can produce the desired results or even better, such designs should be considered. However, the designer is responsible for the designs which are made with respect to stormwater management for any given site. This manual should be used in conjunction with other established manuals and practices. It updates the Stormwater Management Practices Planning and Design Manual (June 1994).

This manual will also be used as a baseline reference document in the review of stormwater management applications for approval under section 53 of the Ontario Water Resources Act as administered by the Ministry of the Environment.

ACKNOWLEDGEMENTS

This project was initiated and funded by the Ontario Ministry of the Environment. It was also funded and supported by the Government of Canada's Great Lakes Sustainability Fund (GLSF), Credit Valley Conservation, and the Toronto and Region Conservation Authority. The following agencies and staff participated on the steering committee.

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The consultants for the project were Aquafor Beech Ltd. (Dave Maunder) and Marshall Macklin Monaghan Ltd. (Doug Andrews) in association with the Centre for Watershed Protection and Environmental Water Resources Group.

EXECUTIVE SUMMARY

Stormwater management is required to mitigate the effects of urbanization on the hydrologic cycle including increased runoff, and decreased infiltration, of rain and snowmelt. Without proper stormwater management, reduced baseflow, degradation of water quality, and increased flooding and erosion can lead to reduced diversity of aquatic life, fewer opportunities for human uses of water resources, and loss of property and human life.

Watershed planning integrates environmental and land use planning. Criteria for the protection of water quantity, water quality, habitat, and biota are established to help achieve the goals set for the watershed. Strategies to manage human activities within the watershed are developed to meet protection criteria. A stormwater management strategy may include protection of natural areas, design of communities to reduce stormwater generation, and pollution prevention programs, as well as the stormwater management practices which are the focus of this technical manual.

A combination of lot level, conveyance, and end-of-pipe stormwater management practices are usually required to meet the multiple objectives of stormwater management: maintaining the hydrologic cycle, protection of water quality, and preventing increased erosion and flooding. Lot level and conveyance controls may be classified as storage or infiltration controls. Storage controls are designed to detain stormwater. Although the volume of runoff does not decrease, the risk of flooding is reduced because all the stormwater runoff does not arrive at the stream at the same time. Infiltration controls are necessary for soil moisture replenishment and groundwater recharge. They can achieve water quality enhancement but are ideally suited for infiltration of relatively clean stormwater including rooftop and foundation drainage. Pre-treatment of road drainage is necessary to prevent clogging of a system and to protect groundwater quality.

End-of-pipe stormwater management practices must control the effects of urbanization which remain after preventative techniques and lot level and conveyance measures have been applied. End-of-pipe facilities are usually required for flood and erosion control and water quality improvement, although lot level and conveyance controls can reduce the size of the end-of-pipe facilities required.

Design guidance is provided for individual lot level, conveyance, and end-of-pipe practices. It includes physical constraints to the use of the practices, such as, soil type and depth to groundwater; sizing and configuration; and design details which vary considerably but which may include inlets and outlets, filter media, and distribution pipes. The guidance also includes cold climate considerations and the incorporation of vegetation in design.

Proper maintenance is critical to the successful performance of a stormwater management system. During the first two years of operation, inspections after significant storms will ensure that the system is functioning properly. After this, annual checks may be done to identify maintenance needs. Blockages may need to be cleared from inlets and outlets. Unhealthy vegetation may need to be tended or replaced. The design of stormwater management practices for water quality improvement is based primarily on settling of sediment. Therefore, at some point, accumulated material will need to be removed.

A preferred stormwater management system will be selected based on its cost, as well as other factors such as technical feasibility, effectiveness, and social acceptability. The overall cost must include capital, operating, and maintenance costs. Information provided may be used for preliminary estimates of cost. However, refinement of estimates to reflect site-specific considerations will be required.

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1.0 INTRODUCTION

1.1 History of Manual

In June 1991, the Ministry of the Environment published a report entitled *Stormwater Quality Best Management Practices*. The report documented experience with structural and non-structural Stormwater Management Practices (SWMPs) and concluded that they should be implemented in conjunction with new urban development and redevelopment.

Guidance, and a procedure for selecting appropriate SWMP types, was provided. The report stated, however, that "integrated watershed planning is the preferred means of defining uses of the receiver and hence the basis for SWMP selection." Recognition of the importance of watershed and subwatershed-based planning has continued to grow since the release of the 1991 study.

The Ministry of the Environment initiated the development of a *Stormwater Management Practices Planning and Design Manual* which was published in June 1994. Stormwater management has evolved considerably in Ontario since 1994; therefore, the Ontario Ministry of the Environment together with the Government of Canada's Great Lakes Sustainability Fund, Credit Valley Conservation, and other agencies undertook a project to update the manual. Furthermore, the 1994 manual focused more on water quality and there was an interest by various parties to produce a more integrated approach that incorporated water quantity and erosion considerations. This manual provides further development and update of key components.

Regarding the intended use of this document, it is worth emphasizing points made in the preface.

> *The "state-of-the-art" of stormwater management has been rapidly evolving and this manual is one step in this evolutionary process. The manual provides technical and procedural guidance for the planning, design, and review of stormwater management practices. It is important that the manual be viewed as a tool for understanding the performance requirements of stormwater management projects and not as a rulebook for all stormwater management solutions.*
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> *technologies. Where the designer can show that alternate approaches can produce the desired results or even better, such designs should be considered. However, the designer is responsible for the designs which are made with respect to stormwater management for any given site. This manual should be used in conjunction with other established manuals and practices. It updates the Stormwater Management Practices Planning and Design Manual (June 1994).*
>
> *This manual will also be used as a baseline reference document in the review of stormwater management applications for approval under section 53 of the Ontario Water Resources Act as administered by the Ministry of the Environment.*

Changes incorporated in the 2003 Manual were based on feedback received from a variety of sources including: a User's Survey given to individuals who obtained the 1994 Manual or attended a seminar in 1994; a questionnaire, circulated in 1997 to provincial agencies, conservation authorities, municipalities and other storm water management professionals; and, two workshops. A Steering Committee, comprised of stakeholders from a variety of agencies, reviewed material and provided input and direction throughout the development of the manual.

The key components (topics) which have been added, expanded, or updated in this manual include:

- providing an overview of the impacts of urbanization on the hydrologic cycle and stream ecosystems;
- addressing the evolution of the watershed planning process and implications for the design process;
- incorporating water quantity, erosion control, water quality protection, and water balance principles into the selection and design of Stormwater Management Practices (SWMPs);
- documenting the performance of SWMPs that have been monitored;
- incorporating design considerations for cold climate conditions for SWMPs;
- providing information on SWMPs such as sand filters, bioretention filters, wet swales and hybrid wet ponds/wetlands;
- writing a chapter on infill projects;
- updating the operation and maintenance chapter;
- providing design examples for SWMPs;
- updating material relating to planting strategies and the function of plant material in the design of SWMPs;
- providing an appendix on assessment methodology for retrofitting SWMPs together with an example; and
- providing an appendix which deals with integrated planning for stormwater management.

Further consultation with stakeholders took place through the posting of a revised draft on the Environmental Bill of Rights registry in 1999-2000. The final manual reflects the changes made as a result of the comments received.

There will be a transition period in which the guidance in the new manual will begin to be applied. Reasonable efforts should be taken to minimize any disruption to on-going projects during the transition period.

1.2 Manual Outline

Provided below is a brief overview of each chapter, as well as the key components that have been added to this version.

Chapter 1 – Introduction
Provides an introduction to the Manual, outlining its history and topics which have been introduced, further developed, or updated, as well as a brief overview of the contents within each chapter. Also provided is an introduction to the potential impacts of urbanization on hydrological aspects of the natural environment.

Chapter 2 – Environmental Planning
Describes the environmental planning process and its relationship with the municipal land use planning and approval process. It describes the types of environmental planning studies that may be undertaken, the deliverables provided by each study, and methods for integrating planning for stormwater management. It also notes that if there are insufficient environmental studies completed to define environmental conditions to set environmental goals/objectives and targets, and to provide a basis for selection and design of SWMPs, additional work (Chapter 3) will be required prior to the initiation of the design process (Chapter 4).

Chapter 3 – Environmental Design Criteria
Presents general environmental design criteria to be used in lieu of criteria that would normally be available from the environmental planning process. Information outlining the required areas of consideration is provided, as is guidance which should be used in order to identify potential problems and provide a reasonable level of impact mitigation. Criteria are presented for water balance, water quality, erosion control/geomorphology, and water quantity.

Chapter 4 – Stormwater Management Plan and SWMP Design
Provides design guidance on SWMPs. The guidance has been updated to reflect recent experience and supplemented where new techniques are available. Quantity control SWMPs have been added. The SWMPs have been grouped as lot level and conveyance controls and end-of-pipe controls. New SWMPs introduced include storage controls, wet swales, hybrid wet pond/wetlands, perimeter sand filters, and bioretention filters.

Chapter 5 – Infill Development
Discusses the challenge of applying stormwater management practices for small, infill sites within developed areas. Approaches and techniques are discussed, including use of off-site systems as an alternative.

Chapter 6 – Operation, Maintenance, and Monitoring
Topics include the need for maintenance, the tasks to be completed, as well as the frequency with which the activities should be undertaken. This advice remains essentially as provided in the 1994 manual although information on vegetation (natural succession) and hard-bottom forebays has been updated.

Chapter 7 – Capital and Operational Costs
Remains largely unchanged from the advice provided in the 1994 Manual. Costs have been updated.

Chapter 8 – References

1.3 Hydrological Effects of Urban Development and their Impacts on Ecosystems

Hydrologic Cycle

The hydrologic cycle describes the continuous circulation of water between the oceans, atmosphere, and land. Water is supplied to the atmosphere by evapotranspiration, which includes evaporation from all water, snow, vegetation, and other surfaces, plus transpiration from plants. It is returned to the land through precipitation. Within the hydrologic cycle, water may be stored by vegetation, snowpacks, land surfaces, water bodies, saturated subsurface zones, and unsaturated subsurface zones/soils. Water may be transported between these storages via overland runoff, streamflow, infiltration, groundwater recharge, and groundwater flow, among other processes (Figure 1.1).

Humans interact with the hydrologic cycle by extracting water for agricultural, domestic, and industrial uses, and returning it as wastewater discharges. Urban development may also interfere with the natural transfers of water between storage components of the hydrologic cycle.

For any system with defined boundaries (e.g., a watershed), a water balance may be used to describe the hydrological cycle. More specifically, the water balance provides for an accounting of water transfers across the system's boundaries over some time period. Any difference between inflows to the system and outflows from the system during this time period must be balanced by a change of storage within the system.

Changes to the Hydrologic Cycle/Water Balance

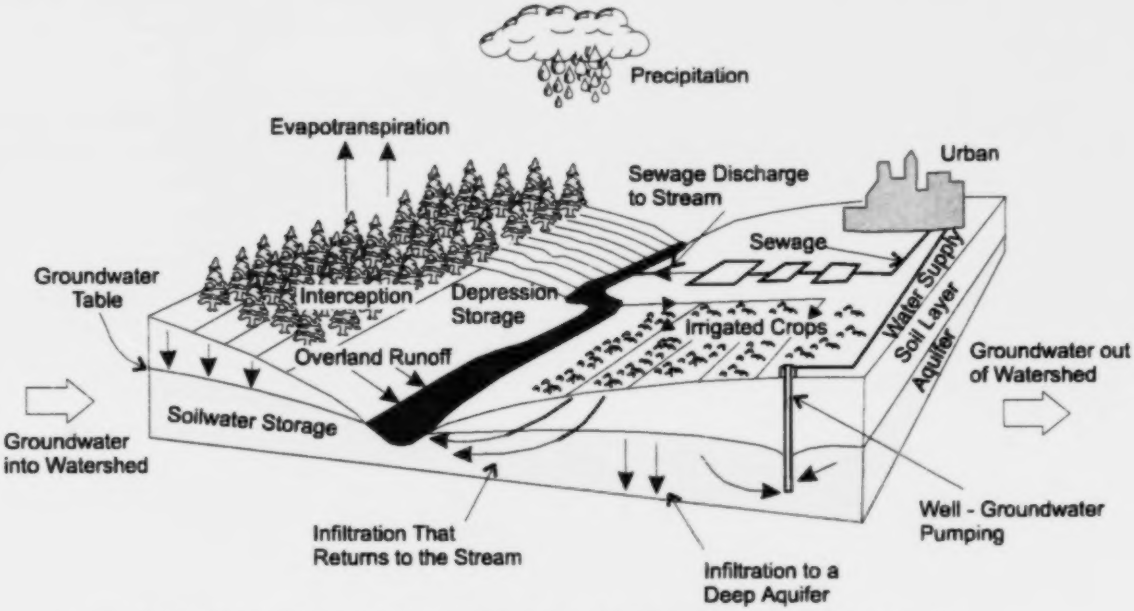
A major consequence of the increase in impervious area which accompanies urbanization is an increase in direct runoff and a corresponding decrease in infiltration. Table 1.1 (WEF, 1998) illustrates the changes in hydrological components that result from developing a forested area. Urbanization also results in decreased evapotranspiration. The net effect of conventional development practices on an urban stream is a dramatic change in the hydrologic regime of the stream.

Effects include:

- an increase in the magnitude and frequency of runoff events of all sizes;
- delivery of more of the stream's annual flow as surface storm runoff rather than base flow or interflow; and
- increases in velocity of flow during storms.

The decrease in infiltration that occurs with urbanization reduces soil moisture replenishment and groundwater recharge. In Ontario, a significant proportion of domestic and agricultural water supplies are from a groundwater source. Groundwater is also the source of stream baseflow which is important for sustaining aquatic life.

Figure 1.1: Hydrological Cycle


Precipitation
Evapotranspiration
Urban
Sewage Discharge to Stream
Sewage
Groundwater Table
Interception
Depression Storage
Irrigated Crops
Water Supply
Soil Layer
Aquifer
Overland Runoff
Groundwater out of Watershed
Soilwater Storage
Groundwater into Watershed
Well - Groundwater Pumping
Infiltration That Returns to the Stream
Infiltration to a Deep Aquifer

Source : After, M. L. Davis, D. A. Cornwell. **Introduction to Environmental Engineering,** 1991.

Definitions:
Overland runoff – water that travels over the ground surface to a channel
Streamflow – movement of water via channels
Groundwater flow – movement of water through the subsurface
Infiltration – penetration of water through the ground surface
Groundwater recharge – water that reaches saturated zone

The preservation of the natural hydrologic cycle, to the greatest extent possible, will not only maintain groundwater recharge so as to reduce baseflow impacts, but it will reduce the potential for flooding and erosion, and hence, the size and cost of stormwater infrastructure. Therefore, it is one of the primary goals of stormwater management.

Changes in Stream Response to Storm Events

Urban floods differ from those in natural basins in the shape of flood hydrographs (Figure 1.2), peak magnitudes relative to the contributing area, and times of occurrence during the year. The imperviousness of urban areas along with the greater hydraulic efficiency of urban conveyance elements cause increased peak streamflows but also more rapid stream response. Summer floods resulting from high intensity thunderstorms are more common in urban areas. Infiltration and evapotranspiration are much reduced at this time of the year under developed conditions.

The goal of stormwater management is to minimize the risks of loss of life and property damage due to urban floods.

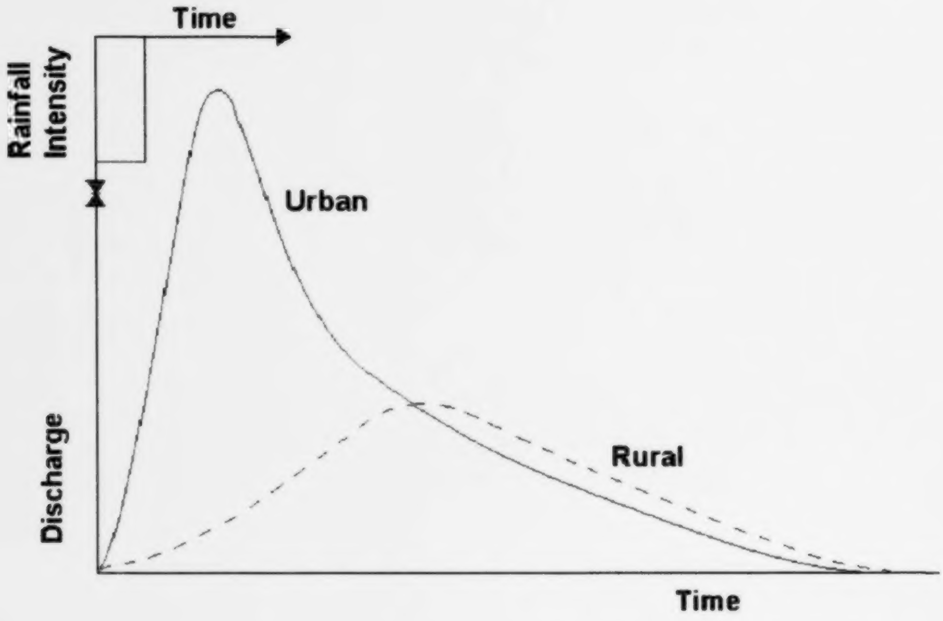
Table 1.1: Distribution of May to November Rainfall for Forested and Urbanized Areas

	Forested Areas		**Urban Areas with 40% Impervious Cover**	
Item	**Depth (mm)**	**% of Total Depth**	**Depth (mm)**	**% of Total Depth**
May to November Rainfall	515	100.0	515	100
Interception Storage and Depression Storage on Impervious Areas	342	66.5	235	45
Infiltration	155	30.0	100	20
Runoff	18	3.5	180	35

Note: Sandy soils assumed.

Source: Modified by Chris Doherty of Environmental Water Resources Group

Figure 1.2: Flood Hydrographs for Urbanized and Natural Drainage Basins (Watt et al, 1989)



Changes in Stream Morphology

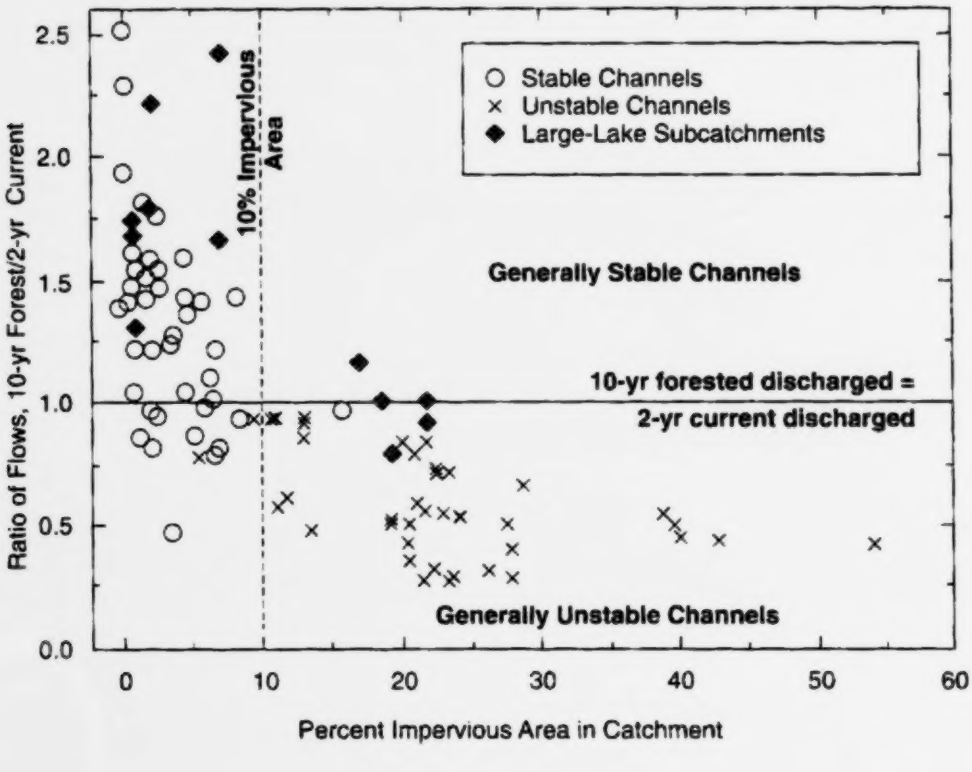
Stream channels in urban areas respond and adjust to the altered hydrologic regime that accompanies urbanization (WEF, 1998). The severity and extent of stream adjustment is a function of the degree of watershed imperviousness as well as the stream type. Examples of stream adjustments include:

- increased stream cross-sectional area to accommodate higher flows;
- significant downcutting of the stream channel;
- increased sediment loads in the stream because of increased instream erosion as well as watershed inputs;
- modification of the streambed (typically the grain size of channel sediments shifts from coarse-grained particles to a mixture of fine- and coarse-grained particles); and
- changes in characteristics such as location and meander pattern in response to stream crossings by roads and pipelines.

There may also be direct modifications of streams, such as straightening and/or lining, by humans to "improve" drainage and reduce flooding risks.

A critical issue is the level of development at which stream morphology begins to change significantly. Research models developed in the Pacific Northwest (U.S.) suggest that a threshold for urban stream stability exists at approximately 10% imperviousness of a watershed (Figure 1.3) (Booth and Reinelt, 1993). Watershed development beyond this threshold consistently results in unstable and eroding channels. The severity and extent of stream adjustment is a function of the magnitude of the change in the sediment-flow regime and the resistance of the channel materials to erosion. The goal of stormwater management is to protect the aquatic ecosystem, as well as the stream's aesthetic and recreational values, by maintaining a stable fluvial system.

Figure 1.3: Channel Stability as Function of Imperviousness (Booth and Reinelt, 1993)



Changes to Water Quality

Deterioration of urban stream water quality is associated with two phases of urbanization. During the initial phase of development, an urban stream can receive a significant pulse of sediment eroded from upland construction sites, even if erosion and sediment controls are used. In the second phase of urbanization, the washing off of accumulated deposits from impervious areas during storms becomes the dominant source of contaminants. This manual focuses on mitigation of the effects of the second phase of urbanization. Guideline B-6: Guidelines for Evaluating Construction Activities Impacting on Water Resources (MOE, 1995) provides guidance for mitigation of the effects of construction activities.

Urban stormwater runoff may contain elevated levels of suspended solids, nutrients, bacteria, heavy metals, oil and grease, and pesticides, as well as sodium and chloride from roadsalt. Table 1.2 shows the concentrations of selected constituents of stormwater runoff compared to the Provincial Water Quality Objectives (Aquafor, 1993). Urban runoff may also cause increased water temperatures.

Table 1.2: Comparison of Urban Stormwater Runoff Concentrations with Provincial Water Quality Objectives

Parameter	Units	PWQO	Observed Concentrations
Fecal coliforms	CNT/dL	—	10,000 - 16E6
SS	mg/L	—	87 - 188
TP	mg/L	0.03	0.3 - 0.7
TKN	mg/L	—	1.9 - 3
Phenolics	mg/L	0.001	0.014 - 0.019
Al	mg/L	—	1.2 - 2.5
Fe	mg/L	—	2.7 - 7.2
Pb	mg/L	0.025	0.038 - 0.055
Ag	mg/L	0.0001	0.002 - 0.005
Cu	mg/L	0.005	0.045 - 0.46
Ni	mg/L	0.025	0.009 - 0.016
Zn	mg/L	0.030	0.14 - 0.26
Cd	mg/L	0.0002	0.001 - 0.024

The change in the sediment load of a stream is one of the key factors affecting channel erosion but elevated levels of suspended solids, including both organic and inorganic matter, may have a number of other effects on a receiving water. Increased turbidity interferes with photosynthetic activity by reducing light penetration. Solids in suspension may clog gills and interfere with fish feeding, and the deposition of sediment may cover spawning areas and smother benthic communities. Organic matter exerts an oxygen demand and may severely depress the levels of dissolved oxygen in the receiving water. In addition, several other stormwater contaminants are commonly associated with solids.

The priority of stormwater management with respect to water quality has been control of suspended solids. However, many of the SWMPs can successfully remove other stormwater contaminants as well. Measures that prevent or minimize releases of contaminants that may be carried to streams by stormwater are, of course, preferable to treatment options.

Groundwater quality may also be affected in urban areas, and care must be taken that the stormwater management controls chosen do not contribute to groundwater degradation.

Changes in Aquatic Habitat and Ecology
The ecology of urban streams and other aquatic habitat is shaped and molded by extreme shifts in hydrology, geomorphology and water quality that accompany the development process. Stresses on the aquatic communities of urban streams and other water resources are often manifested as:

- a shift from external (leaf matter) to internal (algal organic matter) stream production;
- a decline in aquatic habitat quality;
- a reduction in diversity in the fish, plant, animal and aquatic insect communities in the stream;
- a loss of sensitive coldwater species;
- a destruction of freshwater wetlands, riparian buffers and springs; and
- a decline in wetland plant and animal community diversity.

1.4 Environmental and Municipal Land Use Planning

There is a recognition that a more holistic approach is required to mitigate the impacts of urbanization. This has led to the ecosystem approach for development where environmental planning such as watershed and subwatershed planning is done to provide important information to key decision points in the municipal land use planning process. Chapter 2 describes the typical deliverables from each type of environmental plan such as the watershed plan, subwatershed plan, environmental management plan and the stormwater management plan. The plans should provide direction to proponents of development regarding the impacts of the levels and types of development and the management actions required.

The intent of watershed and subwatershed plans is to prepare goal-oriented strategic plans which will allow urban development to occur while protecting the natural ecosystem functions. Watershed-wide policies or management programs are proposed which are mainly oriented towards conservation and preservation such as agricultural restrictions, buffer strips, salt management, topsoil preservation, wildlife linkages, wetland preservation, natural areas preservation, and forest preservation. Watershed and subwatershed plans look at the cumulative effect of development and do not go down to the level of detail needed for design.

The subwatershed plan evaluates the integrated effect of land use scenarios (development, terrestrial linkages preservation, stream buffer preservation, environmentally sensitive/significant area preservation), and urban SWMPs on objectives related to water balance, stream erosion, water quality, temperature, baseflow, flooding, fisheries habitat and aquatic life. For example, a subwatershed plan may set tributary-based targets for peak flows, baseflow and water quality and specify the aggregate levels of stormwater control. Decisions made at the subwatershed plan have direct bearing on the type of development and acceptable SWMP types and performance level at the stormwater management plan level. The results will govern SWMP selection and design for urban development.

An environmental management plan summarizes the findings of the previous plans and is done on a tributary subcatchment boundary or Secondary Plan boundary or a portion thereof. The smaller scale analysis done for an EMP allows for more refined and specific deliverables than a subwatershed plan. EMPs should be of sufficient detail such that all remaining environmental and/or SWM work may be completed as conditions of the Draft or Site Plan stage. Preliminary SWM designs are done at this stage.

The more detailed SWM plan is prepared at the urban subdivision level to meet the conditions and targets set at the Draft or Site Plan stage. The SWM plan is carried out under private proponency and submitted to the review agencies for comment and approval. The SWM planning is integrated with environmental site planning which includes subdivision planning, site planning and engineering, landscape design, architectural and building design, and local street design. It includes the detailed design of SWMPs.

Subdivision/site planning extends the ecosystem approach from watershed planning to the actual layout of the development. Site planning techniques refer to the layout of development and development standards imposed by the local municipalities. It is a fundamental determinant of the overall change in the hydrologic cycle for a given development. The way a development is planned, and the specific design criteria adopted by the planner or engineer, can have a great impact on the level of success achieved by the stormwater management measures which are implemented.

1.5 Urban Stormwater Management Practices

Table 1.3 introduces the types of stormwater management controls which will be discussed in this manual, and their suitability for mitigating the impacts of urban development. Lot level and conveyance controls include those that are applied at the individual lot level, those which form part of the conveyance system, and controls which may serve multiple lots but are only suitable for small drainage areas (< 2 hectares). End-of-pipe controls receive water from a conveyance system and discharge to a receiving water. They are typically the facilities used to service numerous lots or whole subdivisions.

The term "treatment train" is used to describe the combination of controls usually required in an overall stormwater management strategy to ensure that:

- groundwater and baseflow characteristics are preserved;
- water quality will be protected;
- the watercourse will not undergo undesirable geomorphic change;
- there will not be any increase in flood damage potential; and ultimately
- that an appropriate diversity of aquatic life and opportunities for human uses will be maintained.

Lot level and conveyance controls are required to maintain the natural hydrologic cycle to the greatest extent possible. End-of-pipe facilities are usually required for flood and erosion control and water quality improvement, although lot level and conveyance controls can reduce the size of the end of pipe facilities required.

An overall stormwater management strategy may incorporate broader solutions to stormwater management than the practices described in this manual. These broader solutions may be elements of community design (e.g., reduced pavement width, compact building forms) or preventative measures that can be taken by individuals, businesses, or government agencies (e.g., use of safer alternative products and methods, street cleaning, spill prevention and control).

Table 1.3: Stormwater Management Practices

SWMP	Water Balance	Water Quality	Erosion	Water Quantity
Lot Level and Conveyance Controls				
Rooftop storage	○	○	○	●
Parking lot storage	○	○	○	●
Superpipe storage	○	○	○	●
Reduced lot grading	●	•	•	○
Roof leader to ponding area	●	•	•	○
Roof leader to soakaway pit	●	•	•	○
Infiltration trench	●	●	•	○
Grassed swales	●	•	•	•
Pervious pipes	●	●	•	○
Pervious catchbasins	●	•	•	○
Vegetated filter strips	●	•	•	○
Natural buffer strips	•	•	•	○
Rooftop gardens	○	•	•	○
End-of-Pipe Controls				
Wet pond	○	●	●	●
Artificial Wetland	○	●	●	●
Dry pond	○	•	●	●
Infiltration basin	•	●	•	○
Filters*	○	●	○	○
Oil/grit separators*	○	•	○	○

● High Suitability • Medium Suitability ○ Low Suitability

* *Water Quality suitability is highly dependent on sizing and by-pass design.*

Source: Aquafor Beech Ltd., Wet Weather Discharges to the Metropolitan Toronto Waterfront, prepared for the Metropolitan Toronto & Region Remedial Action Plan, 1993

2.0 ENVIRONMENTAL PLANNING

2.1 Introduction

As the field of environmental planning has evolved, a variety of documents have been produced to assist practitioners. These documents include the trilogy of Watershed Planning documents (MOEE, MNR, 1993), as well as others (Science and Technology Task Group, 1995; CVC, 1996). Collectively, these documents provide:

- A rationale for considering watersheds as the natural and logical boundary for environmental and land use planning;

- Direction with respect to the types of environmental studies that are required and the necessary expertise; and

- A process designed to assist agencies and practitioners in working together to balance social, environmental and economic needs, using an ecosystem approach.

The objective of this chapter will be to draw upon the above noted documents together with other publications (including approximately 100 subwatershed plans that have been completed within Ontario) in order to:

- Describe the environmental planning process and its relationship with the municipal land use process;

- Discuss, in general terms, the process used and the typical deliverables from each type of environmental study;

- Provide direction as to how the deliverables from existing studies may be used to assist and simplify the SWMP design process; and

- Provide direction as to how stormwater management may be integrated with the environmental and municipal land use planning processes.

2.2 Environmental and Municipal Land Use Planning

2.2.1 General

Within each municipality, there will be differences as to how municipal land use and environmental planning will be undertaken. It is, therefore, not possible to define a process that is applicable to all municipalities for all types of studies. The intent of this section, therefore, will be to describe environmental and municipal land use planning in general.

Figure 2.1 illustrates general inter-relationships between municipal land use and environmental planning. The agencies that are typically involved with the review of documents at each phase are listed. Also provided (Figure 2.2) is a graphical representation of each of the four types of environmental studies. This graphic provides an overview of the scale of each study and, therefore, the degree of detail (or type of product) to be produced.

Provided below is a brief description of each of the environmental planning studies (i.e., Watershed Plan, Subwatershed Plan, Environmental Management Plan and Environmental/Stormwater Management Plan) that may be undertaken. The objective of this section is to indicate what each type of plan typically provides. Several agencies have produced procedural manuals for environmental planning studies.

There may be other terms used for each type of plan in different jurisdictions. It is important to note the level of detail for each type of plan. In some cases the plans or elements of the plans may be combined.

2.2.2 Managing for Uncertainty

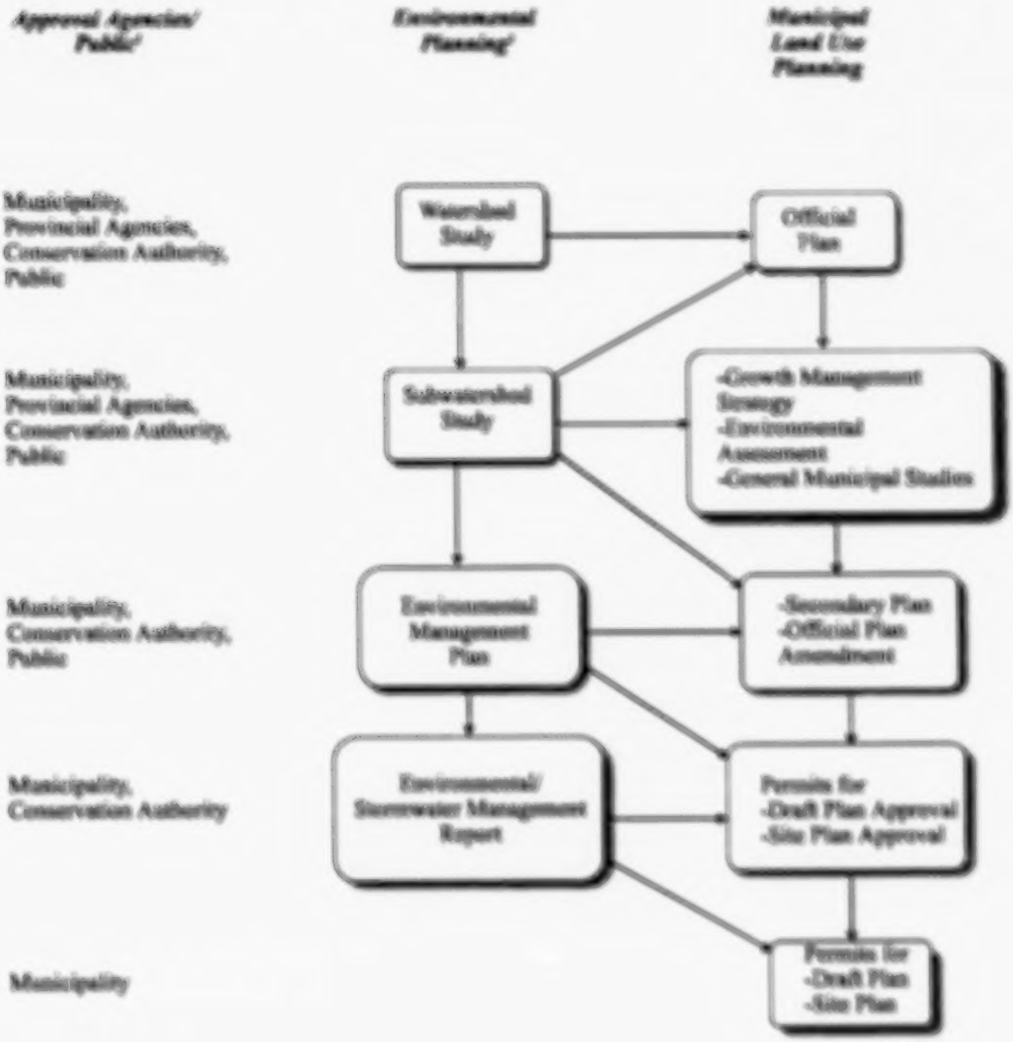
Implicit in the watershed and subwatershed planning process is the uncertainty associated with deriving environmental form, functions and linkages, and forecasting future impacts based on land use and land management changes. The concept of adaptive environmental management recognizes this uncertainty, acknowledges learning through experimentation, and promotes using the knowledge gained to direct changes in management activities.

In order to measure the success of a watershed and subwatershed plan, it is imperative that clearly defined goals and objectives be formulated, and that a monitoring plan be implemented that provides the information necessary to evaluate whether the goals and objectives are being met. Changes to the recommended plan may be necessary if the results of the monitoring show that the objectives are not being achieved. As the process moves from the watershed, subwatershed and environmental management plans to the stormwater management report, management decisions can be adapted to better fit the local circumstances of each subwatershed, tributary and plan of subdivision. This approach allows for flexibility, consensus building and joint learning by managers of the environment, proponents for change and the public.

2.2.3 Required Expertise

Undertaking environmental planning studies in concert with municipal land use studies will reduce the amount of work to be undertaken and streamline the review process, provided the proper expertise and representation from the agencies, public and consultants is available at the appropriate time (Figure 2.1).

Figure 2.1: General Relationship Between the Environmental Planning and the Municipal Land Use Planning Process


Approval Agencies/ Public¹
Environmental Planning²
Municipal Land Use Planning
Municipality, Provincial Agencies, Conservation Authority, Public
Watershed Study
Official Plan
Municipality, Provincial Agencies, Conservation Authority, Public
Subwatershed Study
-Growth Management Strategy
-Environmental Assessment
-General Municipal Studies
Municipality, Conservation Authority, Public
Environmental Management Plan
-Secondary Plan
-Official Plan Amendment
Municipality, Conservation Authority
Environmental/ Stormwater Management Report
Permits for
-Draft Plan Approval
-Site Plan Approval
Municipality
Permits for
-Draft Plan
-Site Plan

1 Approval agency / public involvement will vary from jurisdiction to jurisdiction.
2 For a given jurisdictional area one Environmental Planning component would generally be associated with one Municipal Land use component. Multiple arrows leading from the Environmental Planning component to the Municipal Land Use component signify different approaches which are used in different jurisdictions.

Most stakeholders are involved at the initial stages where key decisions on broad issues (e.g., establishing goals/objectives, incorporating public input, establishing an implementation process) are required. Once consensus has been achieved for issues at the subwatershed or watershed scale, fewer agencies are generally required to review subsequent documents (e.g., EMPs, Environmental/SWM Plans).

As the process has evolved, so too have the disciplines required to undertake and review the studies. The expertise required for each study is dependent upon the level of detail required. However, in general, both the proponent and review agency require expertise in:

- surface water resources;
- groundwater;
- aquatic resources;
- water quality;
- terrestrial ecology;
- municipal engineering;
- fluvial geomorphology; and
- environmental and land use planning.

2.2.4 Watershed Plan

As is illustrated in Figure 2.2, watershed plans deal with the area drained by a major river (e.g., Credit, Grand, Rideau, Don, Thames) and its tributaries. The study area defined by the natural drainage boundaries of the watershed is considerable and usually at least 1,000 km^2.

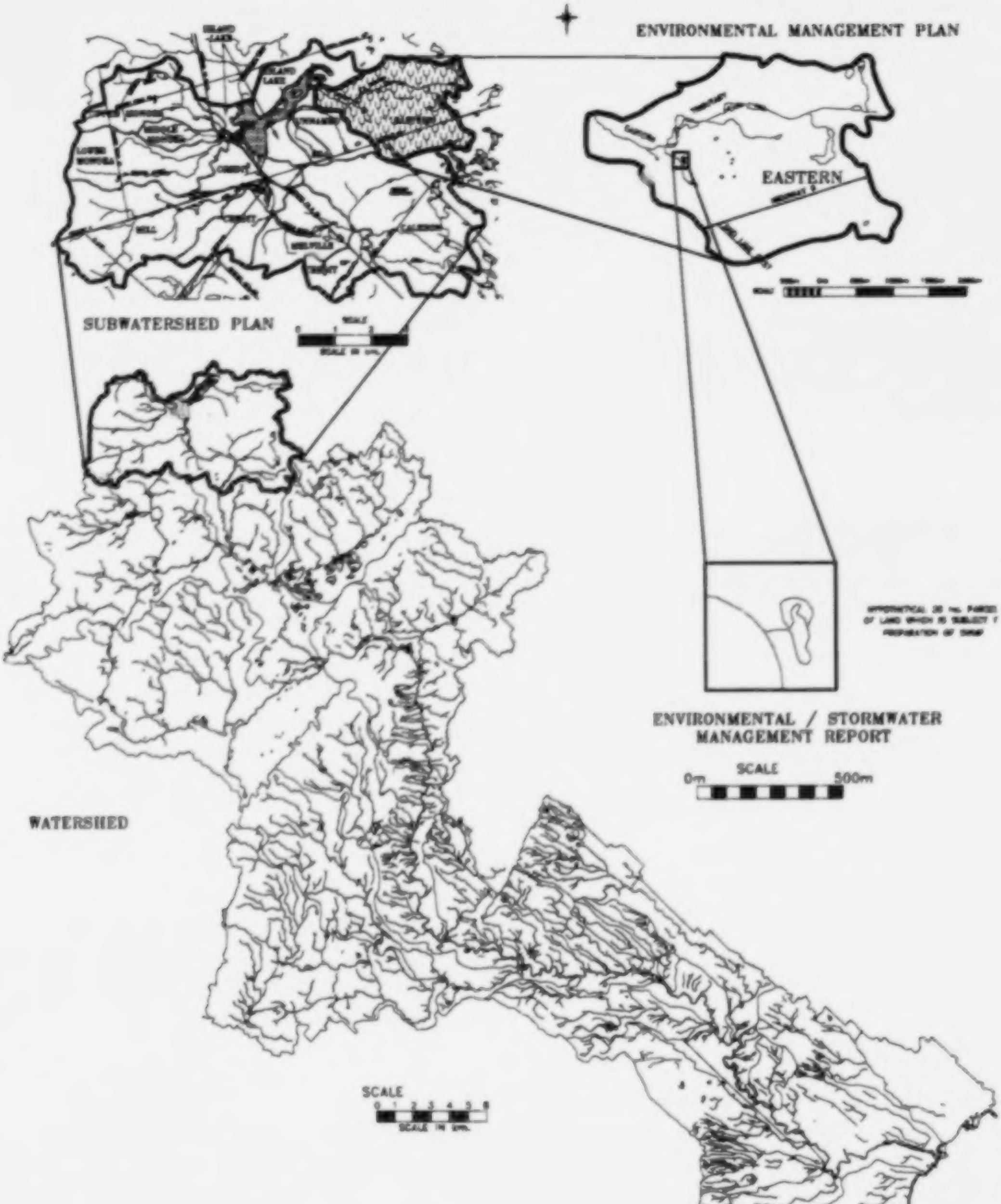
A watershed plan is generally of value for addressing environmental issues associated with studies on the scale of an Official Plan. The study may be used to provide an overall picture as to how land use practices have influenced environmental resources or how land use changes should take place without causing adverse impacts to the watershed resources. Along with the strategic direction, the document may contain recommendations on implementation and funding, as well as resource management goals and objectives.

The watershed plan also provides direction for subsequent subwatershed studies. In this regard, information pertaining to resources, constraints, sources of contamination, key issues, resource goals and environmental targets may be provided.

2.2.5 Subwatershed Plan

The area under consideration in a subwatershed plan is typically 50 to 200 km^2. In some jurisdictions it is the first document produced. It is at this stage that all relevant agencies and the public participate. With a broad range of input being received and with the proper technical and implementation steps being undertaken, it should be possible to carry out subsequent studies at a

Figure 2.2: Graphical Representation of Environmental Studies


ENVIRONMENTAL MANAGEMENT PLAN
EASTERN
SUBWATERSHED PLAN
ENVIRONMENTAL / STORMWATER MANAGEMENT REPORT
SCALE
0m
500m
WATERSHED
SCALE
0 1 2 3 4 5 6
SCALE IN KM.

much smaller scale (typically at a tributary or Secondary Plan level). The key steps of a subwatershed plan are shown in Figure 2.3, and are briefly described below.

Collect Background Information

The initial step involves the collection of background information. This may include relevant reports and base maps, as well as historical information that may be of value for defining change within the subwatershed over time. The relationship of the subwatershed study to a watershed plan, or other urban drainage, land use and planning studies should also be defined.

Establish Existing Environmental Conditions

A series of technical studies will be undertaken to establish existing environmental conditions. In some cases, the information will be available from previous studies, while in others some field work will be required. Typical component studies that need to be undertaken include:

- **Surface water resources**, including an evaluation of the water budget, baseflows, and peak flows as well as flood line assessment;
- **Hydrogeology**, including definition of geologic conditions; groundwater flow patterns and recharge/discharge areas; location, capacity, and quality of aquifers; and quantification of existing well usage;
- **Surface water quality**, including characterization of water quality constituents for dry and wet weather conditions;
- **Fluvial geomorphology**, including classification of streams with respect to their stability and sensitivity to land use change;
- **Terrestrial resources**, including characterization of resources such as wetlands, woodlots, landforms and specially designated natural areas; and
- **Aquatic resources**, including fish and macroinvertebrate (aquatic insect) inventories.

These component studies will identify the location, areal extent, present status, significance and sensitivity of the existing natural environment within the subwatershed.

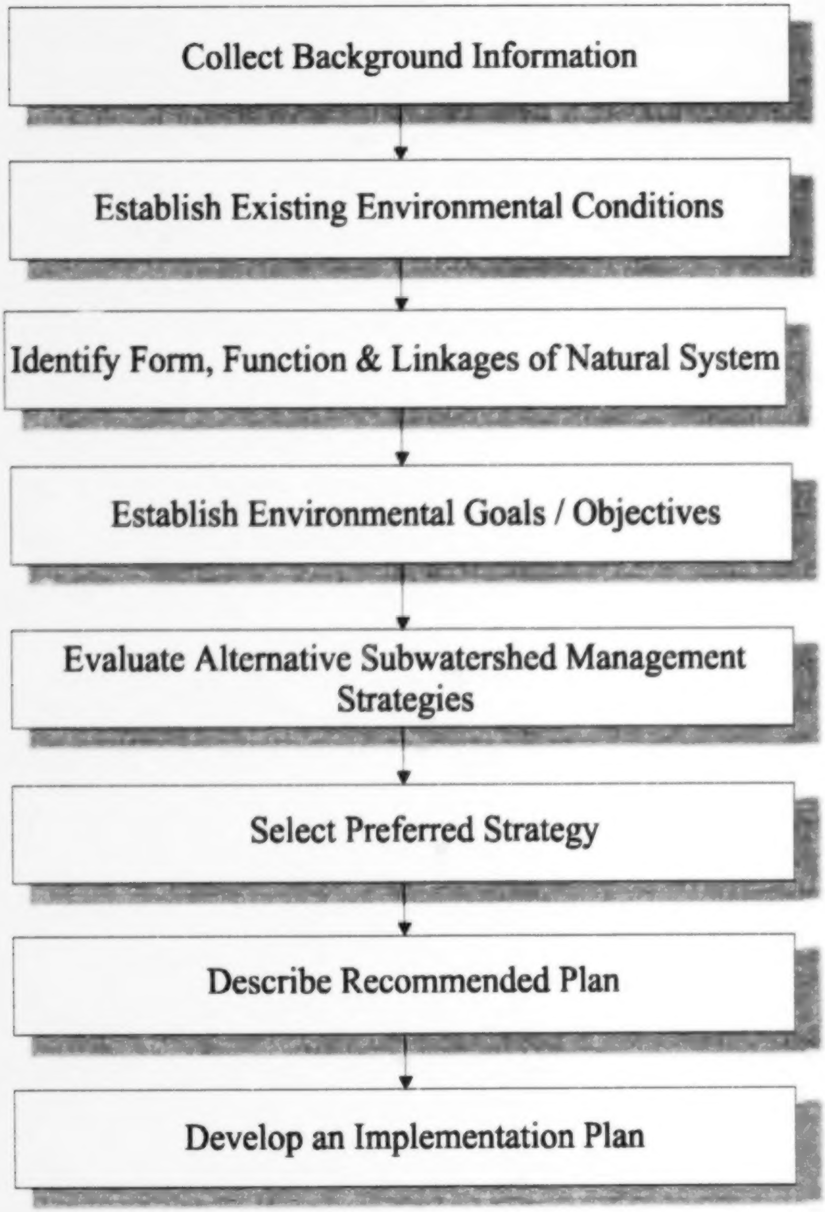
Identify Form, Function and Linkage of Natural Systems

The inter-relationships between the different resources need to be defined. This is done, in part, via an overlay process using GIS and by assessing the technical information collected in the previous step. The inter-relationships (e.g., the presence of a high quality fishery in catchment areas containing good riparian cover, high recharge and wetlands) are important in order to identify key attributes which need to be protected or restored.

Establish Environmental Goals and Objectives

An understanding of the existing conditions and linkages will assist stakeholders in defining environmental goals and objectives for the subwatershed. These goals and objectives will be used

Figure 2.3: Components of a Subwatershed Plan


Collect Background Information
Establish Existing Environmental Conditions
Identify Form, Function & Linkages of Natural System
Establish Environmental Goals / Objectives
Evaluate Alternative Subwatershed Management Strategies
Select Preferred Strategy
Describe Recommended Plan
Develop an Implementation Plan

as a basis for developing alternate subwatershed management strategies, and will also likely be used in subsequent, more detailed studies.

Develop and Evaluate Alternative Subwatershed Management Strategies
Alternative management strategies will be developed for the subwatershed which involve various combinations of management options, including SWMPs. When implemented collectively, the management options contained in each alternative are sufficient to meet the defined environmental goals and objectives for the subwatershed.

Select Preferred Strategy
A preferred strategy is selected from the alternatives based on criteria which may include:

- public acceptance;
- cost;
- technical feasibility;
- ability to meet defined goals and objectives;
- potential of the strategy to enhance the environment; and
- impact of the strategy on future land uses.

Describe Recommended Plan
The Recommended Plan for the Preferred Strategy will specify areas for protection, restoration and/or enhancement. It will also identify areas which are developable, or developable subject to further study. Typically, the Recommended Plan will also provide a general description of each SWMP type, together with items relating to technical considerations, social, economic and environmental benefit, environmental criteria, land requirements/impact and approximate cost.

Implementation Plan
This part of the study will provide direction with respect to the steps which are necessary to implement the recommended plan. Components of the implementation plan include:

- subwatershed plan administration;
- rehabilitation/restoration opportunities; and
- implementation considerations for each SWMP, including:

    - land use planning considerations;
    - cost;
    - review agency;
    - funding;
    - education/stewardship opportunities;
    - monitoring requirements;
    - future studies/initiatives;
    - timeframe for review/update of plan; and
    - community participation.

As was stated previously, one of the objectives of a Subwatershed Plan is to provide information that may be used in subsequent, more detailed studies. One product that has been produced in several subwatershed studies is a Fact Sheet. Table 2.1 shows a fact sheet for a tributary within the subwatershed. Fact Sheets are often used to summarize key resources and targets, and may be used as a basis to establish the appropriate studies and measures to be undertaken, typically at the Official Plan Amendment or Secondary Plan stage.

2.2.6 Environmental Management Plan

The Environmental Management Plan (also referred to as an Environmental Impact Report, Environmental Area Plan or Master Environmental Servicing Plan) is typically carried out prior to consideration of Draft Plan Approval. The relationship of the EMP to other studies is shown in Figures 2.1 and 2.2.

Whereas watershed and subwatershed plans have been ongoing since the late 1980s, EMPs are a relatively new study. The deliverables from an EMP, therefore, are not as consistent from jurisdiction to jurisdiction. The typical area under consideration for an EMP is 2 to 10 km^2. The boundaries for the EMP may match the tributary subcatchment boundary or Secondary Plan boundary or a portion thereof.

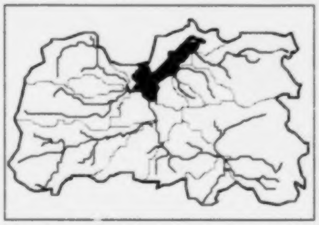
With respect to the level of detail of the EMP, one of the objectives of the Subwatershed Plan was to provide sufficient detail so that future work would not be required beyond the tributary or Secondary Plan level. In an analogous manner, the EMP should be of sufficient detail that individual subdivision plans may proceed pending the completion of the EMP. Provided below is an overview of the components to be undertaken in an EMP.

Review Existing Information
All existing background information should be reviewed to confirm the level of detail and suitability for use in the EMP. Information will be available from the subwatershed study, as well as other studies completed within the study area. Relevant information may include:

- field inventories of woodlots, wetland communities and other vegetation communities;
- land uses for both existing and proposed conditions;
- historical data relating to surface or groundwater quality;
- information relating to aquatic habitat; and
- existing hydrologic modelling and/or floodplain mapping.

Table 2.1: Credit River Subwatershed No. 19 – Eastern Tributary Fact Sheet

Environmental Resources



Groundwater Resources
- significant baseflow contribution in the upper reach
- moderate to high susceptibility to contamination

Surface Water Resources
- 83.0 ha drainage area
- intermittent baseflow, limited to upper reaches
- no floodplain mapping completed, no known flood prone areas
- small online ponds, not used for stormwater management

Aquatic Resources
- Level 2 riparian corridor
- Level 3 riparian corridor
- 50-70% of riparian corridor in natural vegetation
- moderately tolerant to very tolerant warm water aquatic community

Terrestrial Resources
- 15 features in high recharge or on riparian corridors (11 < 4 ha, 2 @ 4-16 ha, 2 @ 36-100 ha)
- 1 secondary terrestrial corridor for protection/revegetation
- 5 other features (4 < 4 ha, 1 @ 4-16 ha)
- linkage to Orangeville Reservoir ESA No. 58

Stream Morphology
- Stable natural channel systems with isolated impacts from rural land use practices (cattle crossings)

Existing Land Uses
- primarily low intensity agricultural uses

Environmental Protection Targets

Groundwater
- protect groundwater discharge areas
- maintain existing stream discharge areas; minimum 5% average annual daily flow
- maintain or enhance on-site recharge and discharge volumes
- maintain current groundwater quality

Surface water quantity and quality
- peak flows requirements to be determined
- maintain baseflow volume
- maximum temperature 30°C
- Total Phosphorus $\leq$ 0.1 mg/L (annual average for dry/wet weather conditions)
- Suspended Solids $\leq$ 20/200 mg/l (annual average for dry/wet weather conditions)
- Dissolved Oxygen 4.0 mg/L minimum at all times
- *E. coli* 100/1,000 cts/100 ml (geometric mean conditions for dry/wet weather conditions)

Aquatic Communities
- Class 3 tolerant warmwater aquatic communities
- Class 3 moderate tolerant aquatic community
- Level 2 riparian and valley corridor
- at least 5% of stream reaches pool-riffle dominated
- minimum pool depths: 0.2 m
- 30-50% of banks with woody riparian vegetation

Table 2.1: Credit River Subwatershed No. 19 – Eastern Tributary Fact Sheet (cont'd)

Terrestrial Communities
- maintain water levels and surface area of marsh features
- protect all Class 1 features
- use native species for revegetating corridors
- select vegetation community types based on soil moisture/fertility, flooding/water table regime and composition of adjacent communities (also see Tables 7.4.2 to 7.4.4)

Stream Morphology
- maintain post-development shear stress conditions on channel systems to pre-development conditions by controlling bankfull frequencies and durations
- maintain dynamically stable, natural channel systems
- balance sediment regime
- minimize fine sediment loads from runoff

Key Best Management Practices (refer to Section 5.0 of the main report for details)

- stormwater runoff volume controls utilizing large open space areas and centralized end-of-pipe facilities to infiltrate clean runoff for maintaining water budget and stream bankfull flow conditions.
- stormwater runoff flow controls and detention ponds for flood control, water quality control and stream morphology protection
- development exclusion in all Class 1 features
- minimize the potential for contaminant spills and minimize applications of de-icing agents, fertilizers, pesticides, etc.
- stream restoration for reaches impacted by livestock access, see Section 7

Future Study Requirements

General
- Environmental Management Planning Studies on a tributary basis (may include studies listed below)
- headwater tributary assessments

Groundwater Resources
- detailed hydrologic studies on baseflow and water supply impacts resulting from future urban growth and urbanization and for the implementation of stormwater management facilities
- investigate area for potential groundwater development

Surface Water Resources
- Hydrologic and hydraulic studies for land use changes
- prepare floodplain mapping

Aquatic Resources
- identify riparian and valley corridor width based on functional characteristics and existing vegetation features

Terrestrial Resources
- EIS required adjacent to all Class 1 features
- EIS required within/adjacent to all Class 2 features
- Naturalization plans for all Class ½ riparian/valley corridors and terrestrial regeneration areas

Stream Morphology
- detailed fluvial geomorphology study to determine design criteria for managing bankfull flow conditions and for natural channel design works, as required

Source: Aquafor Beech Ltd., Environmental Planning for the Credit River Headwaters Subwatershed No. 19, prepared for CVC, 1997.

Define Existing Environmental Conditions and Establish Constraint/Opportunity Mapping
Existing information should be utilized and further study should be focussed on filling gaps and enhancing the understanding of key resources. Detailed constraint and opportunity mapping will be prepared which describes the environmental resources, together with the associated land requirements.

The smaller scale analysis done for an EMP allows knowledge to be refined and more specific deliverables (Table 2.2) to be provided, as compared to a Subwatershed Study.

Establish Preferred Environmental and Stormwater Management Strategy
Typically, impact assessments are carried out (i.e., how does the hydrologic cycle change, how will increases in metals loadings impact fisheries) for the proposed land use. Alternative management strategies are developed, each including a series of environmental and stormwater management practices. Collectively, the management practices will ensure that the environmental resources are protected, restored or enhanced in accordance with defined goals and objectives. Comparative evaluation of the strategies will lead to the selection of the preferred alternative.

Preparation of the Environmental Management Plan
An Environmental Management Plan which summarizes the findings of the previous steps should be prepared. It is important to ensure that the plan highlights the deliverables and is of sufficient detail such that all remaining environmental and/or stormwater management work may be completed as conditions of the Draft or Site Plan stage. Key deliverables may include:

- a map illustrating areas to be protected/restored, together with proposed development patterns;
- preliminary design of restoration/enhancement measures, e.g., proposed cross-section of terrestrial/aquatic/recreation corridor;
- summary of findings from the EIS;
- location sizing and preliminary design of all SWMPs, together with drainage areas;
- identification of areas where special consideration is required at the subdivision plan stage, e.g., areas requiring grading limits and tree preservation planning;
- detailed description of steps to be undertaken at the subdivision plan phase;
- preliminary sediment control plan; and
- operation and maintenance considerations.

Other deliverables that may be presented, depending upon the level of detail and preference/requirements of the agencies include:

- land use patterns;
- open space connections and linkages;
- top of bank delineation; and
- major/minor system routes.

Table 2.2: Detailed Component Studies of an Environmental Management Plan

Surface Water and Groundwater • identification of flow characteristics for all watercourses • preparation of a water budget • preparation of floodplain mapping • description of groundwater resources, including identification of recharge/discharge areas • description of water quality • description of the relationship between ground and surface water and dependencies between these features and the surrounding terrestrial and aquatic resources
Aquatic Resources/Stream Morphology • definition of the morphology of each stream, including the general condition and assessment of erosion sites • characterization of the aquatic resources within each watercourse • determination of headwater streams that provide a function and, therefore, need to be protected • determination of stream corridor boundaries necessary to protect/enhance the functions provided • description of the relationship and dependencies between these features and the surrounding surface water, groundwater and terrestrial resources
Terrestrial Resources • characterization of ecologic resources including woodlots, wetlands and other vegetative features • determination, through existing information or the completion of an EIS, of vegetative features which are to be retained and/or enhanced together with appropriate setbacks, appropriate adjacent land uses, as well as definition of features which may be replaced or are not required to be protected • description of the relationship and dependencies between these features and the surrounding surface water, groundwater and aquatic resources

2.2.7 Environmental/Stormwater Management Report

The Environmental/Stormwater Management Report is generally prepared in order to meet conditions set at the Draft Plan or Site Plan stage. The plan provides details with respect to the proposed environmental and stormwater management measures, and is usually submitted with grading and erosion plans and site servicing plans.

Several documents detail the requirements of these reports. The specific requirements will vary from municipality to municipality. The components of the study will also vary depending upon whether an EMP and/or subwatershed study has been completed.

Typical deliverables from an Environmental/Stormwater Management Report include:

- detailed design of SWMPs, including connections and outfalls;
- detailed design of environmental restoration works (e.g., stream protection works);
- delineation/confirmation of constraint boundaries (e.g., significant woodland, top-of-bank, geotechnical hazard area);
- sediment/erosion control plans;
- detailed reports relating to geotechnical and water resources;
- major/minor systems;
- delineation of grading limits and tree preservation planning;
- revegetation/landscape plans;
- access routes, disposal areas for operation/maintenance; and
- landscape features including trails, benches and other recreational and interpretive amenities.

2.2.8 Review Process

Figure 2.1 shows the stakeholders who are involved at each phase of the review process. The review process, as shown, involves a majority of the stakeholders at the initial stages where key decisions involving a number of issues (e.g., establishing goals/objectives, incorporating public input, establishing an implementation process) are required. Once consensus has been achieved for a majority of issues at the subwatershed or watershed scale, then fewer agencies are generally required to review subsequent documents (e.g., EMPs, Environmental/SWM Plans).

2.3 Integrated Planning for Stormwater Management

Good planning which has regard for the need for stormwater management at the outset, combined with a recognition of the ecological attributes and functions of the watershed, provides the fundamental basis for achieving stormwater quality and quantity improvement efficiently and cost effectively. At this scale, stormwater management opportunities afforded by the physiographic and ecological features of the watershed can be identified, and capitalized upon. Areas where soils are permeable and suited to infiltration, existing vegetation communities which can function as biofilters and landforms that are naturally conducive to the implementation of detention initiatives will all be identified along with a suite of other characteristics which may be preserved or modified to achieve stormwater management objectives. The watershed planning approach, which is now ingrained within the municipal planning process, ensures that the important features and other interrelated factors are identified and understood at a regional scale. As a result, stormwater management opportunities afforded by the existing natural heritage features and functional systems are identified early in the process. This helps ensure that these opportunities will not be overlooked or lost when stormwater management initiatives are implemented at a site-specific scale.

2.3.1 Watershed Scale Solutions

At the watershed scale, the framework for the generation of a stormwater management system, which is integrated with the environment, is established. Effective stormwater management solutions to be explored at a watershed scale are rooted in a recognition of the value and potential function of the existing physical and ecological characteristics of the region. The following examples are provided:

- Identification of areas of high soil permeability presents opportunities to incorporate infiltration-based SWMPs which can provide groundwater recharge (provided that the quality of the stormwater is adequate);
- Existing vegetation affords opportunities to intercept rainfall and moderate rates of discharge;
- Areas of rich organic soils afford the opportunity to enhance water quality through biofiltration;
- Natural depressions in landform provide opportunities to implement detention type SWMPs while minimizing the requirement for extensive earthworks, and the associated costs; and
- The configuration of stream corridors, vegetation units and other terrestrial features may present opportunities to locate SWMPs to improve linkages between existing features, enhancing the connectivity of terrestrial habitats and affording benefits that extend beyond the limits of the watercourse.

Watershed planning is the most effective means of ensuring that responsible stormwater management solutions that are fully integrated with the regional ecosystem are achieved. It is also important to consistently apply this ecosystem-based approach at the community and site-specific levels to ensure that stormwater management opportunities afforded by site and context are identified and capitalized upon.

2.3.2 Community Scale Solutions

The term 'community' in this context is used to describe a level of planning at which the design for a 'community' is being resolved and may include the preparation of a secondary plan, a design plan for a development or a plan of a subdivision. Subdivision/site planning is described in more detail in Appendix A. At this scale, where land use patterns, road networks and open space systems are being defined, a range of opportunities to implement integrated solutions is afforded.

Consistent with the principle of watershed planning, solutions should be founded on a recognition of the natural heritage attributes of the site, including landform, vegetation, water resources, soil types and aquatic and terrestrial habitats. The system of open spaces is recognized as the most obvious opportunity to integrate stormwater management solutions successfully within the design of a community. However, there are numerous other opportunities to achieve stormwater management objectives as well as other multiple ecological, functional, social and economic objectives at this scale.

Open Space System

The open space system should be defined with the vision of establishing a network of natural features and complementary and compatible land uses which will be the spine or centerpiece of the community. The design for the open space system should be conceived with the goals of preserving existing resources, enhancing the overall ecological integrity of the site and providing recreational, social and educational benefits. The integration of stormwater management initiatives as components of the open space system contributes to the realization of these goals by increasing the physical area of available open space, enhancing terrestrial and aquatic habitat diversity and enhancing recreational and educational opportunities. The following are examples of techniques which have been applied to integrate stormwater management initiatives and enhance the open space network:

- Incorporation of wet ponds and wetlands within active or passive parks as ecological or recreational features;
- Integration of ponds and wetlands with school blocks to provide outdoor environmental education opportunities;
- Design of pond systems to replicate a new valley corridor and extension of a tributary of an existing river system;
- Design of a series of wet ponds as an aesthetic feature or entrance feature within a community;
- Creation of a wetland as an extension of an existing forest community;
- Integration of playfields within the basin of a dry detention pond;
- Design of subsurface storage and/or infiltration systems beneath playfields within parks or school yards;
- Installation of infiltration galleries beneath walkways as part of a recreational trail system; and
- Incorporation of a constructed peat biofilter as an educational amenity within a park.

The breadth of opportunity to implement innovative solutions is expanded when the open space network is comprised of a cohesive system of natural areas, parks, school yards, stormwater management facilities, trails and linkages rather than a collection of isolated and disjointed parcels. The former affords a wider range of possibilities to integrate different types of SWMPs in series. Opportunities are lost when proposed stormwater management facilities are relegated to 'leftover' spaces rather than being considered as integral components and potential amenities within a development. A well executed open space network, which includes stormwater management facilities, has been proven to enhance the marketability of a development by establishing a character for the community and increasing the range of available amenities. In considering the design of the open space network, emphasis should be placed on establishing a seamless system of spaces with complementary uses built upon the existing natural features of the site.

Municipal Transportation Network

Stormwater management solutions can be applied to the municipal network of roads within a community. Although much more constrained by requirements related to servicing, alignment, safety and gradient than the open space system, road right-of-ways present the opportunity to implement simple, cost-effective and beneficial stormwater management solutions. References for guidance on Ministry of Transportation (MTO) requirements for provincial highways are mentioned in Section 3.1.

Reduced Pavement Width
A reduction in the width of the paved cross section of the road affords benefits such as:

- reduction in impervious area;
- opportunities to implement SWMPs such as grassed swales, within the right-of-way; and
- enhanced potential to increase canopy cover through street tree planting.

Implementation of Grassed Swales
Not only effective in terms of stormwater quality improvement, the implementation of grassed swales can result in substantial savings in cost when compared with conventional storm sewer servicing. Grassed swales also provide benefits related to snow storage and groundwater recharge where appropriate soil conditions exist.

Porous Pavement
Although not regarded as durable and sustainable for universal application, porous pavement, in the form of granular or precast concrete unit paving can be used in appropriate areas. Such areas include the shoulder of the road to provide a transition between the travelled surface and the grassed swale, in the centre of cul-de-sacs or in parking lanes. When used in these applications, porous pavement provides practical benefits as well as benefits related to water quality improvement and aesthetics.

Pocket Detention Storage
Within the road right-of-way, there are a number of small areas well suited to the implementation of pocket detention facilities or biofilters. Cul-de-sac islands, medians, boulevards, roundabout islands, and in the case of limited access routes, leftover land within interchanges, should be considered as potential sites to detain stormwater and settle out pollutants. These areas can be paved or landscaped to integrate them into the aesthetics of a streetscape or character of the development. Although the sizes of these facilities are limited, collectively, significant stormwater management benefits can be achieved.

Tree Planting

Increasing canopy cover in the urban context is a simple, effective means to intercept rainfall before it comes into contact with the ground and becomes runoff. In addition, through the process of transpiration, trees can extract moisture from the subsurface and discharge it into the atmosphere. Studies have shown that a mature willow tree can transpire approximately 750 litres of water during a single summer day. Increased planting of large canopy trees in the vicinity of streets, parking lots and other impervious surfaces may contribute to reduced rates of runoff in addition to affording benefits related to the production of oxygen, temperature reduction, habitat enhancement and aesthetics.

Lot Configuration and Grading

Lot configuration and grading are key factors in determining the extent to which lot level controls such as roof leader disconnection, vegetated filter strips and depression storage can be implemented as well as their effectiveness. Lot layout and grading should be defined with an emphasis not only on achieving maximum yield but also with the objective of maximizing the potential to implement lot level controls effectively.

Built Form

Compact building forms such as townhouses, fourplexes, and clusters make available more open space and, consequently, greater opportunities to implement effective landscape-based stormwater management solutions than more conventional detached residential forms. The resultant increase in open space afforded by compact building forms provides the following stormwater management benefits:

- reduction in impervious area;
- the potential to implement a series of smaller centralized SWMPs rather than a single, large end-of-pipe facility; and
- better opportunities to integrate stormwater management initiatives within the development.

Although built form will largely be dictated by market demand, the viability of compact built form options should be explored as a component of the integrated design process.

The most successful plan for a community is one that is developed using an open-minded approach that is focussed on defining a vision which recognizes the value of existing site resources and integrates environmental, social and functional objectives within a cohesive system.

2.4 Subwatershed Plans and Cumulative Impacts

Subwatershed planning and other planning exercises on a catchment basis provide the step-by-step method for evaluating the impacts of different forms and degrees of development on receiving waters. In some tributaries or subwatersheds draining to sensitive aquatic features, impacts may accumulate over time (due to the time for urbanization to be completed) and over space and the effectiveness of mitigating measures may be uncertain. To maintain the integrity of ecosystems, essential planning tools include scientific data documenting the effectiveness of mitigating measures and the consideration of cumulative effects.

2.4.1 Alternative Plans and Cumulative Impacts

In order to evaluate the management options and different types of SWMPs, it is necessary to formulate alternative plans which use various combination of options. All types of SWMPs are used in the formulation of alternative plans. Situations arise where a problem is attributable to a specific source and can be addressed through a specific option. This will, however, be the exception, rather than the rule. Most problems originate from a variety of sources and require a variety of management actions. Concern for stream temperature impacts on fisheries may, for instance, require protection of groundwater discharge areas and sources, restoration of stream canopy, creation of in-stream refuge areas, and use of particular stormwater SWMP types in urban development. No single option can solve the problem and it is necessary to examine the effects of the complete suite of options. The individual options may be the responsibility of different groups, both private and governmental, and it is implicit in the evaluation and assessment that all parties will implement those options which are their responsibility.

There is a second important factor in the formulation of alternative plans for evaluation, and that is the assessment of cumulative impact. Land use change produces both direct and immediately observable impacts and more gradual impacts which may build up to critical levels as growth proceeds, for example, the impacts of imperviousness on benthic organisms and stream stability as outlined in Chapter 1. A range of options aimed at addressing a particular problem may cease to be sufficient, if too much development takes place. The alternative plans must therefore be evaluated not only against each other, but also under a range of growth conditions.

While it is not possible to specify the alternative plans to be considered for watershed plans/ subwatershed plans on a generic basis, a few alternatives which contrast the potential range of impacts should be utilized. The alternatives would normally be evaluated in a cumulative manner with the management options which perform well being carried forward. This allows the assessment of increasing levels of stress being applied to the watershed/subwatershed.

It is important to consider the cumulative effect of SWMPs on the watershed/subwatershed. While these SWMPs are designed to reduce sediment and contaminant impacts from urban runoff, they may alter the water balance or flow regime of water courses. Current practice seeks to more closely maintain the natural hydrology of developing areas through the use of lot level controls to promote infiltration. This is a goal, however, and is very difficult to be completely achieved. The effect of extended discharge on flows, erosion thresholds, and water levels – as they affect aquatic life or wetlands, must be accounted for in developing the watershed/ subwatershed plan.

2.4.2 Evaluation and Assessment

Evaluation and assessment of alternative plans are based on predictions made by forecasting tools. Forecasts are needed for a variety of issues in a subwatershed, including flood evaluation, erosion evaluation, water balance and low flow modelling, groundwater quantity and quality, surface water quality, and ecological assessments. Tools for forecasting flooding are commonly used, and their forecasts readily accepted by professionals and the regulatory community. In fact, tools for addressing water quantity (flooding, low flow, groundwater flow) issues are much more advanced and in wider use than tools for ecological analysis. This SWM Planning and Design Manual has provided an updated methodology (see Chapter 3 and Appendix B) for assessing stream erosion and the effectiveness of management methods, but there remain uncertainties with the methodology which can only be addressed through monitoring studies.

A primary focus of watershed/subwatershed planning is to maintain or improve the ecological health of the watershed as land use change occurs. As a result, it is necessary to evaluate the impacts to the physical system and indicators in terms of their resulting impact on biological systems. The methods used to conduct this assessment range from the use of professional judgment and literature, to the use of empirical models. In the assessment of the mitigating measures to enhance habitat or protect ecosystems, the assessment may be even more based on professional judgment, due to the lack of recognized models or objective field data-sets which document the degree of protection provided.

2.4.3 Cumulative Impacts and Land Use Restrictions

In most watersheds or subwatersheds, there is a limit to which urban development and growth can proceed without causing irreparable damage to natural systems which support the watershed ecosystem. The threshold at which such damage may occur is variable and largely depends on the physical and biological characteristics of the area. The identification and protection of constraint areas and the mitigative measures specified as a part of the criteria to govern development, are methods of extending the development threshold by maximizing the protection provided to the key elements of the watershed. In some instances, however, there may be a need to further limit the cumulative impact of urban development by limiting the levels of imperviousness that are allowable.

In such areas, there are a variety of options ranging from control of development form (e.g., higher density, clustered and buffered development) to complete restriction of portions of the watershed/subwatershed to uses which do not produce a significant change in the hydrologic or hydrogeologic regimes of the watershed. The selection of the appropriate approach is not solely a watershed/subwatershed planning concern, but rather may require an in-depth assessment of the land use needs of the community. It is incumbent upon the watershed/ subwatershed plan to identify the concerns for cumulative impact, to identify the areas which could be subject to land use restrictions and to identify the levels of imperviousness which would be acceptable to avoid undesirable impacts. The watershed/subwatershed plan is not intended to set land use policy but to set the environmental factors to be met by future land use decisions.

3.0 ENVIRONMENTAL DESIGN CRITERIA

3.1 General

Chapter 2 described watershed and subwatershed plans, as well as the more detailed plans which are now often being completed at the Secondary Plan level. The objectives of the stormwater management design criteria typically provided in such plans are to:

- preserve groundwater and baseflow characteristics;
- prevent undesirable and costly geomorphic change in the watercourse;
- prevent any increase in flood risk potential;
- protect water quality; and ultimately
- maintain an appropriate diversity of aquatic life and opportunities for human uses.

These criteria are developed considering the interactions and cumulative effects which may be expected from urban growth. Cumulative impacts refer to the combined effect of numerous single developments.

Urban development without watershed/subwatershed planning is discouraged because of the difficulty in addressing many environmental impacts at a plan of subdivision or site plan level. Where guidance from a watershed/subwatershed plan is not available, approvals may be delayed due to incomplete information requirements which may extend off site and include:

- cumulative impact of urbanization on aquatic resources;
- wildlife corridors;
- natural area linkages;
- surface and subsurface flow paths;
- rehabilitation areas;
- cumulative impact of individual subdivision/site water management practices; and
- visual impacts.

Although development planning using the subwatershed approach is preferred, there will be cases where a development will be allowed to proceed without a subwatershed plan. While there may be many factors which necessitate this, in general it will occur when the scale of the proposed development is small, the overall level of watershed development (i.e., imperviousness) is limited, and the receiving stream is not overly sensitive in terms of aquatic resources, geomorphology or flooding, nor severely degraded in terms of water quality. The proposed development will typically be an infill (surrounded by existing development), a replacement for existing development, an isolated urban development serving a particular need, or an expansion of the urban fringe. In most cases where a development is allowed to proceed without subwatershed planning, the preparation of a subwatershed plan is determined to be cost ineffective (e.g., there is very little foreseen future development) or cost prohibitive in the near future. The decision to proceed without a subwatershed plan must be confirmed with the approval agencies.

In the absence of watershed/subwatershed planning, subdivision/site planning must occur to ensure that the development is planned with due regard to the surrounding environment. Resource mapping, as described in Appendix A, must be prepared since there will not be any commensurate mapping from a subwatershed plan. This chapter provides guidance on establishing stormwater management design criteria to mitigate the effects of urbanization on the water balance, water quality, stream morphology, and water quantity. Although most of the discussion is focussed on end-of-pipe facilities, lot level and conveyance controls should be utilized to the extent possible in order to maintain the pre-development hydrologic regime and reduce the size of the end-of-pipe facilities.

In some cases, stormwater may be discharged to a receiving drainage system that is part of a highway drainage system such as a highway roadside ditch or a highway storm sewer system. In such cases the impacts on these drainage systems may determine the level of stormwater management control required. Constraints can also be placed by the design capacity and impact of an existing highway culvert or bridge located downstream of a development site. Land development proposals may require approvals from the Ministry of Transportation (MTO) before proceeding. Guidance on how to satisfy MTO requirements and on the design considerations and design practices of stormwater management facilities adjacent to highways can be found in the following references:

- MTO Drainage Management Manual, 1997.
- MTO Stormwater Management Requirements for Land Development Proposals, 1999 (www.mto.gov.on.ca/english/engineering/drainage/drainage.htm).

3.2 Water Balance

3.2.1 Modelling

As described in Section 1.3, urbanization may reduce groundwater recharge which in turn may reduce baseflow, leading to the impairment of aquatic habitats, as well as the water available for domestic, agricultural, or other uses. Therefore, it is necessary to predict the effect of urban development on the subsurface portion of the hydrologic cycle.

Ideally, this may be accomplished using a groundwater modelling approach. An analysis may be conducted to evaluate the sensitivity of the system to reduced recharge and how urbanization may ultimately affect water users or aquatic habitats. The benefit of this approach is that the sensitivity of the groundwater system, not only to the quantity of recharge but also, to the spatial distribution of recharge may be examined. Once developed, a groundwater model may also be used to evaluate alternative mitigation techniques.

The utility of a modelling approach however, is highly dependent upon the quantity of data required to characterize the subsurface system (i.e., complexity of hydrogeologic system) and the quantity and quality of data which is available or may be collected. It may not be feasible to satisfy the relatively intensive data requirements for modelling. Modelling used without discretion may lead to poor decisions. It is important to stress that care must be taken not to accept results which cannot be defended because of poor quality input to a model.

3.2.2 Water Balance Methods

In cases in which the available data cannot support more sophisticated approaches, water balance methods are more appropriate for predicting the changes to the hydrologic cycle that may result from urban development. They can be used to determine amounts of water that should be infiltrated to compensate for reductions caused by large paved areas or changes to vegetation.

The water balance method developed by Thornthwaite and Mather (1957) determines the potential and actual amounts of evapotranspiration and water surplus (or excess of precipitation over evapotranspiration). Infiltration factors are used to determine the fraction of water surplus that infiltrates into the ground and the fraction that runs off to nearby streams. Thornthwaite and Mather's method requires monthly or daily precipitation, monthly or daily temperature, latitude of the site, vegetation type, soil type, and a series of tables. The tables define a heat index, potential evapotranspiration, water holding capacity, and soil moisture retention. Snowfall, and alternating wet and dry cycles are included. Soil water holding capacity is dependent upon the soil type, soil structure and the type of vegetation growing on it. The Thornthwaite and Mather water balance method assumes mature vegetation and does not account for growing seasons where evapotranspiration would be less for immature vegetation.

3.2.3 Water Balance Example

Water balances should be calculated on a site by site basis. Table 3.1 shows the results of a water balance for various vegetation covers in different soil types for a basin in southern Ontario with a latitude of 45°. Infiltration factors were calculated for each soil and vegetation type and were determined for rolling land. More details on infiltration factors can be found in "Hydrogeological Technical Information Requirements for Land Development Applications" (MOE, 1995).

The results shown in Table 3.1 were computed using average annual monthly values. More accurate answers would be obtained using monthly recorded precipitation and temperature for a period of 10 to 20 years. Depending upon the quality of other inputs to the method, the accuracy of the water balance results may be further improved if daily precipitation and temperature values are used.

Table 3.1: Hydrologic Cycle Component Values

	Water Holding Capacity mm	Hydrologic Soil Group	Precipitation mm	Evapo-transpiration mm	Runoff mm	Infiltration* mm
Urban Lawns/Shallow Rooted Crops (spinach, beans, beets, carrots)						
Fine Sand	50	A	940	515	149	276
Fine Sandy Loam	75	B	940	525	187	228
Silt Loam	125	C	940	536	222	182
Clay Loam	100	CD	940	531	245	164
Clay	75	D	940	525	270	145
Moderately Rooted Crops (corn and cereal grains)						
Fine Sand	75	A	940	525	125	291
Fine Sandy Loam	150	B	940	539	160	241
Silt Loam	200	C	940	543	199	199
Clay Loam	200	CD	940	543	218	179
Clay	150	D	940	539	241	160
Pasture and Shrubs						
Fine Sand	100	A	940	531	102	307
Fine Sandy Loam	150	B	940	539	140	261
Silt Loam	250	C	940	546	177	217
Clay Loam	250	CD	940	546	197	197
Clay	200	D	940	543	218	179
Mature Forests						
Fine Sand	250	A	940	546	79	315
Fine Sandy Loam	300	B	940	548	118	274
Silt Loam	400	C	940	550	156	234
Clay Loam	400	CD	940	550	176	215
Clay	350	D	940	549	196	196

Notes: Hydrologic Soil Group A represents soils with low runoff potential and Soil Group D represents soils with high runoff potential. The evapotranspiration values are for mature vegetation. Streamflow is composed of baseflow and runoff.

* *This is the total infiltration of which some discharges back to the stream as base flow. The infiltration factor is determined by summing a factor for topography, soils and cover.*

Topography	Flat Land, average slope < 0.6 m/km	0.3
	Rolling Land, average slope 2.8 m to 3.8 m/km	0.2
	Hilly Land, average slope 28 m to 47 m/km	0.1
Soils	Tight impervious clay	0.1
	Medium combinations of clay and loam	0.2
	Open Sandy loam	0.4
Cover	Cultivated Land	0.1
	Woodland	0.2

As shown in the following simple example, Table 3.1 can be used to determine infiltration amounts for varying land uses:

Pre-Development Conditions

The site area is approximately 10.0 ha with pasture type vegetation in fine sand soil. The average annual site infiltration would be approximately 307 mm or approximately 30,700 m^3 (307 mm × 10.0 ha).

Post-Development Conditions

Of the total site area 3.5 ha (35 %) would be converted to impervious area. The infiltration for this area would be 0 mm. The remaining 6.5 ha of the site (65 %) is assumed to be covered with urban lawns (shallow rooted crops) with an average annual infiltration of 276 mm or approximately 17,940 m^3 (276 mm × 6.5 ha). There would be a net reduction in infiltration of 12,760 m^3. If the reduction has a significant impact, then 12,760 m^3, or some portion of it, may have to be infiltrated using SWMPs.

3.3 Water Quality

3.3.1 Criteria Development

During the development of the 1994 SWMP Manual, a review of the existing water quality criteria in Canada and the United States was made. The primary criteria used in most jurisdictions were volumetric (i.e., runoff from a specified design storm was to be captured and treated). In most cases the selected design storm ranged from 12.5 mm to 25 mm. The use of this type of volumetric design storm criteria remains prevalent today, although some jurisdictions have established methods for refining the size of the design event, based on area-specific conditions such as climate or the receiving water body.

An alternate approach to the volumetric sizing of stormwater facilities has been applied in Ontario. Computer modelling of end-of-pipe stormwater management facilities was undertaken to assess the variation in pollutant removal with SWMP type and level of imperviousness. The modelling results were based on many assumptions, primarily related to the proper design of facilities, and the theoretical build-up, wash-off and settling of sediment particles. The approach however allowed the development of volumetric criteria that reflected a twenty year period of climatic record. This meant that the effect of storms in series (i.e., several storms in a few days), event overflows and winter melt conditions were accounted for in selecting the volumetric criteria. It also allowed specification of the volumetric criteria according to some basic characteristics of the different SWMP types (e.g., depth, detention time). An assessment of regional variations in climate indicated that the same volumetric guidelines could be used throughout the province.

The continuous simulation models yielded several useful, theoretical findings:

- The amount of suspended solids settling for a given design storage varies with SWMP type because of their inherent design characteristics. SWMPs therefore require different volumes of storage to provide the same suspended solids removal performance.

- The volume of water in the permanent pool of a wet facility (wet pond, wetland) is more important than the active storage component (that portion of a facility that drains after an event) for suspended solids removal.

- The suspended solids removal performance becomes asymptotic with increasing design storage (there is a limit to storage beyond which there are negligible increases in suspended solids settling).

The variation in performance with SWMP type was explained by the typical configurations of the facilities and the different removal mechanisms. For example, infiltration type SWMPs were assumed to remove 90% to 95% of the suspended solids from water which was infiltrated. This results in a high removal efficiency if the storage is large enough to contain the storm (or polluted portion of the storm). The model only looked at sedimentation, and assumed that re-suspension of previous settled pollutants would not occur. Therefore, wetlands were more effective than wet ponds since they were modelled with a shallower depth.

The importance of the permanent pool was seen to be considerable. The simulations that were conducted indicated that a wet pond without any extended detention storage was still highly effective for solids settling. The results can be explained by the hydraulic operation of these facilities. During a storm, the influent loading is diluted in the permanent pool. Any discharge from the pond during the storm event is therefore diluted (given that the configuration of the pond is appropriately designed). After the storm has subsided there is still a considerable volume of suspended solids which is trapped in the permanent pool and has not settled. These solids have the inter-event times (i.e., 2 to 3 days on average) to settle out in the pond. This combined action of dilution and inter-event settling makes wet facilities efficient.

The diminishing return for large storage volumes can be explained by the frequency distribution of rainfall events. Once the storage exceeds the volume of most small runoff events, the excess storage provides limited benefit. This is particularly true in terms of the permanent pool volume.

The results of modelling led to the development of volumetric criteria which differed in several major aspects from those found in other jurisdictions:

- For wet facilities, the importance of the permanent pool was recognized by specifying a maximum active storage volume (relative to the total volume);

- Different volumetric criteria were specified for the major classes of SWMP to reflect their varying removal efficiencies (which result from their inherent design); and

- Different volumetric criteria were recommended according to the predicted level of long-term sediment removal.

3.3.1.1 Level of Protection

The federal *Fisheries Act* prohibits "the deposit of a deleterious substance of any type in water frequented by fish or in any place under any conditions where the deleterious substance or any other deleterious substance that results from the deposit of the deleterious substance may enter any such water" (subsection 36(3)). Any substance with a potentially harmful chemical, physical (including temperature) or biological effect on fish or fish habitat is considered to be deleterious. The "first-order" impacts of stormwater runoff are primarily related to suspended solids (SS), however, so the design of facilities is usually based on the long-term removal of SS from the stormwater discharge.

The federal *Fisheries Act* does not differentiate between different types of habitat, but Fisheries and Oceans Canada (Fish Habitat Management) does recognize that some habitats are more resilient to perturbation. Based on this, the levels of protection should be chosen to maintain or enhance the existing aquatic habitat. The level of water quality protection given in watershed management plans, fisheries management plans, official plans, official plan amendments, plans of subdivision, site plans, or other environmental management plans should be adhered to when designing stormwater management facilities. In the absence of these plans, it is possible to select the desired level of protection based on the characteristics of the receiving watercourse. However, the decision regarding the level of protection needed should be made based on input from a qualified aquatic biologist. While general guidance is provided below on the level of protection recommended for the different habitat types, the level of protection should be based on site-specific conditions determined through quantification of pre-development suspended solids loadings to receiving waters and the sediment loading characteristics of the receiving waters. This will require examination of the existing receiving water aquatic habitat and its interaction with the surrounding terrestrial habitat through instream sampling, soil type delineation, vegetation cover, and existing aquatic species inventory as required to justify the level of protection.

Three levels of protection are given, with the goal to maintain or enhance existing aquatic habitat, based on the suspended solids removal performance for the different end-of-pipe stormwater management facilities developed in the continuous simulation modelling. Descriptions of the habitat characteristics corresponding to the three levels of protection are given below.

Enhanced Protection

Enhanced protection or greater should be used when sensitive aquatic habitat will be impacted by end-of-pipe discharge. Generally this will include receiving waters that have aquatic communities that have adapted to a low suspended solids environment. Conditions where a minimum of enhanced protection should be used include:

- Areas with high permeability soils (i.e., Soil Conservation Service (SCS) hydrologic classes A and B) conducive to infiltration resulting in low suspended solids loadings from the pre development site;
- Habitat sensitive to sediment and siltation (such as gravel bottom used for bass or brook trout spawning);
- High baseflow discharge areas (such as groundwater upwellings important to brook trout);
- Low upstream sediment loads resulting in clear surface water important to maintaining habitat for sight feeding fish species (such as bass, northern pike, lake trout, and brook trout); and
- Low pre development erosion characteristics (such as dense vegetation, or erosion resistant soils).

Normal Protection

Normal protection can be considered when conditions for enhanced protection do not exist. Example habitats where normal protection may be appropriate include:

- Areas with moderate, natural upstream sediment loads (such as some walleye feeding habitat); and
- Spawning habitat less sensitive to suspended solids loadings (such as aquatic and emergent plant beds used by pike and perch).

If there is no subwatershed plan or fisheries information available on the receiving waters, agencies with fisheries and habitat management responsibilities may require sufficient background study to justify the use of normal protection where there is known potential for sensitive aquatic habitat within a reasonable distance downstream. Responsible agencies should be contacted early in the design process in order to establish a reasonable downstream distance based on specific studies and local conditions. Generally, normal protection will be considered suitable where a stable downstream habitat has adapted to moderate sediment loading.

Basic Protection

Basic protection would only be acceptable where the receiving aquatic habitat is demonstrated to be insensitive to stormwater impacts and has little potential for immediate or long-term rehabilitation. Generally, basic protection may be applied in the following conditions:

- Areas where downstream aquatic habitat has adapted to high suspended solid loadings prior to anthropomorphic changes to the watershed (for example, aquatic habitat conditions that may be found naturally in areas of fine grained soils); and

- Downstream watercourses have been significantly altered (by urbanization or agricultural practices), hardened, or polluted, and there is little short or long-term potential for rehabilitation.

Proponents proposing basic treatment must seek approval from the appropriate agencies with fisheries and habitat management responsibilities with clear rational and site-specific supporting data collected from baseline studies or from existing resource management agency data bases (such as, fishery management plans, watershed management plans, etc.).

Agencies with fisheries responsibilities may also require habitat compensation where stormwater management design impacts are determined to result in harmful alteration, disruption, or destruction of fish habitat as defined in the *Fisheries Act*. Habitat compensation typically involves the replacing of damaged habitat with newly created habitat or improving the productive capacity of other aquatic habitat at or near the area of impact.

The levels of protection are based on a general relationship between the long-term average suspended solids removal of the end-of-pipe stormwater management facilities and the lethal and chronic effects of suspended solids on aquatic life. The levels of protection correspond to the following 'long-term average suspended solids removals' which refer to the removal by the SWM facility of suspended solids from the site runoff for the entire range of rainfall events on that site for a long period of time, at least 10 years. The use of a long-term average is to account for the variability in characteristics of rainfall events.

- *Enhanced* protection corresponds to the end-of-pipe storage volumes required for the long-term average removal of 80% of suspended solids.

- *Normal* protection corresponds to the end-of-pipe storage volumes required for the long-term average removal of 70% of suspended solids.

- *Basic* protection corresponds to the end-of-pipe storage volumes required for the long-term average removal of 60% of suspended solids.

For SWMPs designed with a by-pass, the calculation of long-term suspended solids removal must be based on both suspended solids removal in the facility plus suspended solids by-passed around the facility.

3.3.2 Water Quality Sizing Criteria

The volumetric water quality criteria are presented in Table 3.2. The values are based on a 24 hour drawdown time and a design which conforms to the guidance provided in this manual. Requirements differ with SWMP type to reflect differences in removal efficiencies. Of the specified storage volume for wet facilities, 40 m³/ha is extended detention, while the remainder represents the permanent pool.

Table 3.2 Water Quality Storage Requirements based on Receiving Waters[1, 2]

Protection Level	SWMP Type	Storage Volume (m³/ha) for Impervious Level			
		35%	55%	70%	85%
Enhanced 80% long-term S.S. removal	Infiltration	25	30	35	40
	Wetlands	80	105	120	140
	Hybrid Wet Pond/Wetland	110	150	175	195
	Wet Pond	140	190	225	250
Normal 70% long-term S.S. removal	Infiltration	20	20	25	30
	Wetlands	60	70	80	90
	Hybrid Wet Pond/Wetland	75	90	105	120
	Wet Pond	90	110	130	150
Basic 60% long-term S.S. removal	Infiltration	20	20	20	20
	Wetlands	60	60	60	60
	Hybrid Wet Pond/Wetland	60	70	75	80
	Wet Pond	60	75	85	95
	Dry Pond (Continuous Flow)	90	150	200	240

[1] *Table 3.2 does not include every available SWMP type. Any SWMP type that can be demonstrated to the approval agencies to meet the required long-term suspended solids removal for the selected protection levels under the conditions of the site is acceptable for water quality objectives. The sizing for these SWMP types is to be determined based on performance results that have been peer-reviewed. The designer and those who review the design should be fully aware of the assumptions and sampling methodologies used in formulating performance predictions and their implications for the design.*

[2] *Hybrid Wet Pond/Wetland systems have 50-60% of their permanent pool volume in deeper portions of the facility (e.g., forebay, wet ponds).*

For levels of imperviousness below 35%, required storage volumes may be obtained by extrapolating the values provided in Table 3.2. For levels of imperviousness between those included in Table 3.2, required storage volumes may be obtained by interpolation.

It should be noted that the total drainage area contributing to the facility should be included in sizing (lumped imperviousness or separate calculations for internal and external drainage areas is permissible) in most cases. The exception occurs when an external drainage area is itself controlled by a separate water quality facility (and erosion and quantity control are either not required or provided separately). Modelling studies (Marshall Macklin Monaghan Limited, 1997) indicate comparable combined long-term removal rates for ponds in series and separate parallel ponds. More frequent overflows will occur from the most downstream pond, but this can be compensated for by doubling the water quality active storage volume from 40 to 80 m^3/ha.

The volumetric criteria specified in Table 3.2 address only water quality, not erosion, baseflow or flooding concerns. Furthermore, the criteria were developed based on the removal of suspended solids via settling, and therefore, may not adequately address contaminants which must be removed by other mechanisms.

3.3.3 Results of Monitoring SWMP Performance

In the late 1990s a partnership of government agencies pooled their resources to undertake a series of monitoring studies aimed at assessing the water quality performance of selected SWMPs through the Stormwater Assessment and Monitoring Performance (SWAMP) Program (Meek and Liang, 1998). Most of the facilities monitored did not meet the design guidance provided in this or the previous version of the Manual as they were constructed before this guidance was available. Nevertheless, the results of the monitoring program are of use in assessing the performance of stormwater management facilities.

In addition to the efforts conducted under SWAMP, numerous studies of performance have been conducted both inside and outside of Ontario. Most performance studies in Ontario have been of wet pond or pond/wetland systems. Key results of performance studies, and their implications to SWMP design in Ontario, are summarized below.

- The results of performance studies indicate a fair consistency for most end-of-pipe SWMP types (typically 60-80% suspended solids (SS) removal and 40-50% total phosphorus (TP) removal);

- Extremes in performance are observed in all end-of-pipe SWMP types (from negative performance to 99% removal of SS and TP);

- For wet facilities, the volume of the permanent pool appears to be important. Some facilities with no active storage (i.e., those with permanent pool only) have performed well;

- Greater than anticipated removal rates have been observed in some instances. Flocculent settling may be the mechanism for enhanced removal;

- Dry ponds (i.e., those with no permanent pool) may be more effective than previously credited, when longer detention times can be achieved (e.g., 48 hours); and

- Performance can be enhanced through techniques other than adding volume (e.g., extending the flow path with baffles).

Overall, the results point to an optimistic view of SWMP performance, particularly in retro-fit situations. The results, however, continue to show significant variability from facility to facility. There is not currently a sufficient body of monitoring results to warrant alterations to the volumetric criteria specified in Table 3.2. It is also apparent that many factors other than volume can influence the performance of a SWM facility.

The analysis of the results of performance studies suggests that:

- The current volumetric criteria should be retained;

- There should be greater emphasis on meeting other recommended design criteria (use of forebays, minimum length-to-width ratio, etc.);

- The monitoring of facilities should be continued, but that the emphasis should be shifted to assessing the processes and mechanisms (and associated design elements) that govern performance which may require alternate monitoring techniques (such as dye tracing); and

- The use of more sophisticated settling and flow dynamics models should be investigated, for testing SWMP design characteristics.

3.3.4 Other Considerations

3.3.4.1 Bacteria

Recreational activities which involve water contact (i.e., swimming) may require additional water quality controls depending on the distance between the development and the recreation area, and the contributing drainage area upstream of the recreation location compared to the size of development. In areas where there are no recreational activities involving water contact, wet stormwater management facilities and infiltration techniques adequately control bacterial loadings (faecal coliform, *E. coli*).

In instances where the proposed development is greater than or equal to 10% of the drainage area discharging to a swimming or other recreational area of concern, a subwatershed plan should be undertaken to address the cumulative impact of development.

3.3.4.2 Temperature

Temperature is a major concern in regard to fish and their habitat, especially where discharge is to a cold water stream. Urbanization causes temperature increases in stormwater and ponds can compound this increase since open water will tend to acclimate with the ambient air temperature. Design for temperature mitigation is discussed in Section 4.4. Where temperature is a significant concern it is recommended that the designer consult with the local conservation authority, the federal Department of Fisheries and Oceans (Fisheries and Habitat Management) and the Ontario Ministry of Natural Resources, during the design process.

3.4 Erosion Control/Geomorphology

This sub-section provides an overview of approaches for the design of end-of-pipe (or centralized) Stormwater Management facilities for the control of in-stream erosion potential. The global intent of SWM measures for the control of in-stream erosion potential is the preservation or enhancement of a "stable," sustainable fluvial system and its associated habitat, aesthetic value and education-recreational potential while accommodating development needs. Two design approaches are described:

- a Detailed Design Approach; and
- a Simplified Design Approach.

These approaches incorporate advances in the field of urban geomorphology and stormwater management. In reading the following sections, the following should be kept in mind:

- The processes that control natural channel systems are complex and span a number of disciplines (e.g., geomorphology, biology, engineering). In order to provide an effective approach to designing a stream system that provides or emulates natural stream qualities, the necessary expertise must be available and integrated in the design process;

- The procedure presented for the Detailed Approach is similar to a Nine-Step Protocol developed by the Ministry of Natural Resources and described in "Adaptive Management of Stream Corridors in Ontario." The Nine-Step Protocol focuses on the broader question of stream management, including stream reconstruction; as such the procedures address the same issues and the same science, but the logical order of analysis differs to a small degree in the two procedures; and

- The approaches, as outlined below, have been applied to over 40 watersheds in Ontario, British Columbia, Texas and Vermont. Confirmation of the approaches would be enhanced through implementation of pilot projects, monitoring, assessment and peer review.

Detailed descriptions of the approaches are provided in the Appendices. Appendix B provides a suggested checklist for the Detailed Design Approach, Appendix C provides additional detail regarding the basis for and application of the Simplified Design Approach, and Appendix D is a technical discussion of one approach, the Distributed Runoff Control approach, which could be used for the design of pond outlet structures.

The following section provides a historical review of stormwater management practices pertaining to erosion control and some of the fundamental concepts and recent findings in the field of urban geomorphology.

3.4.1 Geomorphology Concepts

The active channel is that part of the channel which conveys the dry weather flow and flow from frequent precipitation events. Its dimensions are determined through a balance between those forces tending to dislodge and transport boundary materials and those forces tending to resist movement such that the stream is just able to move its sediment load. The forces tending to dislodge and transport boundary materials are referred to as the erosive forces and they are related to the volume and rate at which sediment and water are delivered to the stream.

An increase in erosive forces is one of the potential consequences of urbanization, and uncontrolled runoff. Channels have an innate ability to tolerate some variability in the influx of sediment and water. This threshold varies with the resistance of the boundary materials and the type, density, and distribution of riparian vegetation. However, it has been found that at levels of watershed imperviousness above about 10%, stream channels become unstable and begin eroding (Figure 1.1). Channel enlargement in urban areas is well documented (MacRae, 1996). The degree of enlargement is a function of the magnitude of the change in the sediment – flow regime and the resistance of the boundary materials.

Once a channel has reached its threshold, it begins a three-stage enlargement process. During the first stage, which takes two to three years, the thalweg (the deepest point in a channel's cross-section) adjusts and the bar forms are reworked. These adjustments may go unnoticed although they may have a detrimental effect on aquatic organisms such as benthic macroinvertebrates. During the second stage, the channel may begin to enlarge rapidly. It may take 35 to 65 years for the channel to adjust to the new sediment-flow regime. The increase in the active channel cross-sectional area may be greater than ten-fold. The final stage involves the re-development of the meander form. The amount of sediment from bank erosion transported by the stream during this stage may be more than ten times that generated during the second stage. However, the adjustment may occur over centuries so the rate of change is less dramatic.

It should be noted that erosion is a normal aspect of river behaviour. Channel function involves conveying water and sediment to larger water bodies. The objective of stormwater management is not to eliminate erosion but to maintain a level of stream erosion such that the channel can continue to fulfill its normal function. Too much control over streamflows may reduce the stream's ability to transport its sediment load resulting in a choking of the channel. Conversely, not enough control may result in too much erosive power causing the stream to erode its boundary and enlarge.

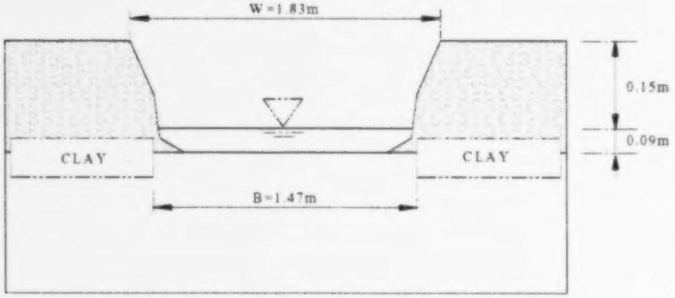
Stormwater management measures developed to control erosion potential, including those adopted in southern Ontario, were based on control of the peak flow rate. Control involved reduction of the post-development peak flow rate for a specified design storm, to the pre-development flow rate for the same storm. The two year storm is frequently adopted as the design event because it has been found to correspond to the bankfull flow stage, when water fills the active channel without spilling out onto the floodplain. The bankfull flow performs the most work, in terms of sediment moved, and consequently, it was believed to be the flow responsible for the shape of the active channel.

The active channel, however, is not formed by any single event. Its form is the consequence of the sum of forces exerted on the boundary by a range of events, from those that partially fill the active channel (from about mid-bankfull) to the bankfull event. Mid-bankfull flows, which rarely occur prior to urbanization, occur frequently following development. The increase in the frequency of their occurrence is such that they may be the events that perform the most work in shaping the channel.

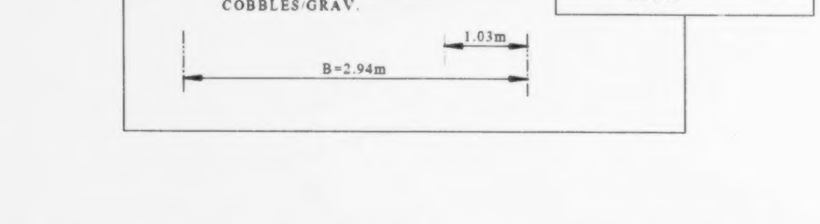
The traditional method adopted for control of erosion potential also does not address the resistance of boundary materials. It assumes the channel is symmetric and the boundary materials are homogeneous (e.g., Enver Creek, Figure 3.1(a)). More typically, channels are asymmetric in form and the boundary materials are heterogeneous deposits (e.g., Serpentine River, Figure 3.1(b)). In the case of Serpentine River, the banks are composed of several different layers of material each of which has unique properties that determine its resistance to erosion. Streams have the tendency to attack the material with the least resistance to erosion. If this material is near the bottom of the bank, the channel will tend to be wider than if the lower materials are more resistant, because the maximum erosive force on the bank is located within the lower third of the bank profile.

Figure 3.1: Channel Forms

a) TYPICAL CROSS-SECTION THROUGH REACH E - ENVER CREEK


W=1.83m
0.15m
0.09m
CLAY
CLAY
B=1.47m

b) REPRESENTATIVE CROSS-SECTION THROUGH REACH F - SERPENTINE RIVER


W=4.73m
LOOSE SANDY SILT
SANDY LOAM
1.22m
SOFT SILTY CLAY LOAM
1.10m
0.175m
FIRM SILTY CLAY LOAM
COBBLES/GRAV.
1.03m
B=2.94m

The traditional approach to control erosion potential fails to recognize the importance of frequent flow events, the heterogeneity of boundary materials, as well as channel stability. In unstable streams, the innate capacity to absorb a change in the flow regime has been diminished. Consequently, the required degree of control may be greater than for stable systems.

A design methodology that overcomes the limitations of the traditional approach for control of in-stream erosion potential would be preferred. The challenge is to balance the need for a comprehensive characterization of the fluvial system with the need for a relatively simple but universal design procedure that may be applied in circumstances where detailed information may not be available.

3.4.2 Detailed Design Approach

The Detailed Design Approach may be:

- selected by the proponent for any development regardless of size and location within the watershed provided technical specialists are available for the completion of the technical assessments; or
- considered more appropriate than the simplified approach given the size and location of the development within the watershed and the sensitivity of the receiving waters in terms of morphology and habitat function.

The principal steps involved in the Detailed Design Approach are listed in Table 3.3. A more detailed outline of these steps is provided in Appendix B.

Table 3.3: Summary of Key Steps in the Detailed Design Approach

Step	Description
1	Clarification of Goals, Objectives and Scope
2	Resource and Land Use Mapping
3	Assessment of Channel System a) Physiographic b) Historical Context c) Regional Data Base
4	Stream Stability-Sensitivity Assessment a) 'Like' Reach Definition b) Representative Reaches c) Rapid Geomorphic Assessment d) Diagnostic Surveys

Table 3.3: Summary of Key Steps in the Detailed Design Approach (cont'd)

Step	Description
5	Constraint-Opportunity Mapping a) Stream Restoration-Protection Options b) Stormwater Management (SWM) Options
6	Stormwater Management Alternatives
7	Design Targets For Control of In-stream Erosion Potential a) Geomorphic Thresholds for Channel Stability b) Geomorphic Criteria for Habitat Protection-Restoration
8	Design Criteria For Control of In-stream Erosion Potential a) Volume Control b) Rate Control
9	Selection of Preferred SWM-Stream Restoration Program
10	Design of Stormwater Management Practices (SWMPs)

Steps 1, 2, and 3 identified in Table 3.3 represent a desktop level analysis of the channel system to establish the framework for the subsequent investigations. The first element pertains to the definition of long-term goals and objectives for the stream channel. These are often linked to habitat issues. For example, a stream regarded as a valuable fisheries resource, which is currently in a degraded state due to agricultural practices, may be targeted for restoration. The investigations pertaining to channel morphology under this scenario may be different than if the channel system was a small, poorly defined, intermittent stream. Consequently, the identification of likely impacts on stream channel morphology associated with development of the basin is closely linked to habitat targets. The sensitivity of the channel system to a disturbance in the flow-sediment regime and the current morphologic state of the channel system are also significant considerations. If the channel has not been affected by a prior disturbance, this element is relatively straightforward. If the channel has been impacted, additional studies may be required to ascertain the degree of impact, the evolutionary state of the channel and its ultimate form. A checklist to deal with these later investigations is provided in Appendix B.

Step 4 involves the collection of parameters characterizing the channel system that are needed to satisfy the scope of the investigations defined in Steps 1 through 3. A 'like' reach approach has been proposed wherein the channel is divided into reaches of 'like' morphology. This approach assumes that the sensitivity of the channel and its mode of adjustment to a disturbance are similar within reaches having similar morphology. A number of representative cross-sections are selected within distinct reaches to characterize the parameters describing the channel system. These data are used to develop hydraulic parameters and geomorphologic relationships modelling the channel system.

This information is subsequently used in the development and design of the SWM measures and channel restoration programs if required.

Steps 5 through 9 deal with the development, assessment and selection of a preferred SWM alternative. The key elements in this component of the investigation are:

- identification of constraints and opportunities;
- the development of SWM design criteria;
- the development of SWM alternatives consisting of suites of SWMPs;
- selection of a preferred SWM alternative; and
- preliminary design of the SWMPs described in the preferred SWM alternative.

The development of design criteria involves the translation of habitat targets into water quality and quantity specifications. The quantity specifications are related to the types of fluvial features required to meet habitat targets and their stability under the proposed development scenario. For example, the preservation of the benthic macro-invertebrate community of a stream requires a specified particle size distribution for the bed materials within a riffle segment. Secondly, the survival of this community requires that these materials not be mobile over more than some specified time period in any year. The stability of these materials can be determined using critical shear stress concepts wherein the threshold for movement and the duration of exceedance of this threshold represent SWM design criteria.

Once the design targets are specified, lot level and conveyance controls may be investigated and integrated into the preferred SWM alternative. Once the level of control provided by these measures has been established, the active storage volume for end-of-pipe facilities required to meet the design constraints may be approximated. Refinements of the estimate of the active storage volume of the facility may be required until the in-stream erosion targets are closely approximated. Finally the rate of outflow is adjusted until the erosion targets are satisfied.

Rate control may be achieved using various approaches. One approach, Distributed Runoff Control (DRC), is outlined in Appendix D. The focus of this method is the preservation of the balance between the erosive and resisting forces about the channel perimeter such that the stream is just able to move its sediment load. As such, the DRC method incorporates many of the elements described in Section 3.4.1 of this report.

Step 10 concerns the development of the detailed design of the SWMPs described in the preferred SWM alternative. It also involves the development of the Implementation Plan. A key aspect of this Plan is the incorporation of an adaptive management approach. This approach acknowledges that the understanding of stream behaviour is incomplete and that a monitoring program is an essential component of any Implementation Plan. Secondly, the Plan identifies stewardship responsibilities for the channel system and fiscal as well as physical mechanisms for the implementation of adjustment procedures or corrective maintenance.

3.4.3 Simplified Design Approach

3.4.3.1 Application of Simplified Design Approach

Application of the Simplified Design Approach requires agreement by both the reviewing agency and the proponent of the development.

The Simplified Design Approach may be adopted for watersheds whose development area is generally less than twenty hectares **AND** either one or the other of the following two conditions apply.

A)
- the catchment area of the receiving channel at the point-of-entry of stormwater drainage from the development is equal to or greater than twenty-five square kilometres;

OR

B)
- the channel bankfull depth is less than three quarters of a metre;
- the channel is a headwater stream;
- the receiving channel is not designated as an Environmentally Sensitive Area (ESA) or Area of Natural or Scientific Interest (ANSI) and does not provide habitat for a sensitive aquatic species;
- the channel is stable to transitional; and
- the channel is slightly entrenched.

The selection criteria are provided in Table 3.4, and explained below.

Table 3.4: Criteria for Selection of Approach for the Design of an End-of-Pipe Facility for the Control of In-Stream Erosion Potential

Parameter	**Criteria**	**Comment/Definition**
Subwatershed/Area Plan	N/A	No Area or Subwatershed Plan exists
Size of Development	$CDA_{DEV} \leq 20$ ha	The Catchment Drainage Area of the Development is generally less than or equal to 20 ha
Headwater (or First-Order) Stream	1st	The stream is a first-order channel according to the Horton classification system using 1:50,000 topographic mapping

Table 3.4: Criteria for Selection of Approach for the Design of an End-of-Pipe Facility for the Control of In-Stream Erosion Potential (cont'd)

Parameter	Criteria	Comment/Definition
Stability Index Value	$SI \leq 0.4$	The channel is classified as stable or transitional according to the Stability Index value computed using the Rapid Geomorphic Assessment (RGA) form (Appendix C)
Entrenchment Ratio	$T \geq 2.2$	The channel is slightly entrenched according to the Rosgen (1996) classification system
Bankfull Depth	$DBFL < 0.75$ m	The bankfull depth is less than 0.75 m in height
ANSI/ESA	N/A	The stream is not considered to be an Area of Natural or Scientific Interest, nor part of an Environmentally Significant Area, and does not provide habitat for a sensitive aquatic species
Riparian Vegetation	Dense	Riparian Vegetation coverage is dense, covers virtually all of the bank area with a root depth which penetrates to or below the low flow water level

Subwatershed/Area Plan: If a Subwatershed or Environmental Management Plan already exists for the proposed development area, then these Plans take precedence.

Size of Development: While the Detailed Design Approach represents a more comprehensive method, a simplified approach was considered useful for small developments where detailed environmental data were not already available.

Stream Order: The channel network can be mapped and the channel divided into segments, according to a hierarchy of orders. Each fingertip, or headwater channel, is designated as a segment of the first order. The order increases at the junction of two first-order segments. A stream of higher order implies a stream of greater geomorphic diversity and greater magnitude in dimension. By definition, first-order streams represent those channels having the smallest dimensions within the watershed.

Stability Index Value: A stable stream has an innate ability to absorb a certain amount of change in the sediment-flow regime before the threshold of adjustment is reached. This tolerance is reduced in streams designated as in "transition" or in "adjustment" according to the Rapid Geomorphic Assessment approach (Appendix C). Because a "zero increase" in erosion potential is difficult to achieve, this tolerance becomes an integral part of the design process. Consequently, a stable stream channel is required for application of the "Simplified Design Approach."

Entrenchment Ratio: The Entrenchment Ratio provides an indication of the flow conveyance capacity of the active channel. A higher flow conveyance capacity means that flows of higher return period (greater flow rate and volume) will be contained within the active channel. Given the likelihood that flow rate and volume will increase as a consequence of development, this additional conveyance capacity translates into higher in-stream erosion potential. In contrast, a less entrenched channel means that flows of higher return period will spill out over the floodplain thereby dissipating the erosive energy. Consequently, a channel of low entrenchment is preferred.

Bankfull Depth: For bank heights of greater than 0.75 m the characteristics of the soil materials (cohesion, particle size and compaction, stratification, etc.) and the root binding effects of vegetation are generally considered to be the controlling and modifying factors, respectively. For these channel systems a stability analysis based on critical shear stress concepts may be required. For bank heights of less than 0.75 m colonized by dense, herbaceous vegetation, the influence of root binding may become dominant. In first-order tributaries having bankfull widths less than 3 m, channel gradients less than 1.5% and mature, dense woody vegetation, the occurrence of Large Organic Debris (LOD) may also control channel form. Consequently, both biological and pedological (i.e., soil) factors may contribute to channel form in first-order channels.

ANSI/ESA: These designations or any other environmentally significant factors that may be identified may require that the Detailed Design Approach be adopted.

Riparian Vegetation: As noted under "Bankfull Depth" above, riparian cover is an important determinant in boundary material resistance to erosion. Riparian cover must be dense and complete to be effective.

3.4.3.2 Overview of Technical Steps

The Simplified Design Approach involves three components:

- a synoptic level geomorphic survey of the stream channel to collect measurements of channel form;
- assessment of the applicability of the Simplified Design Approach for the proposed development; and
- determination of the volume of source control and the active storage volume flow rate for an end-of-pipe facility.

These technical steps are described in more detail in Appendix C.

3.5 Water Quantity

The increase in direct runoff, together with the rapid conveyance of runoff in urban areas, results in increased peak streamflows, particularly during the summer and fall. Winter and spring runoff may not change dramatically because pre-development runoff may be high due to frozen or saturated soils. In contrast, little runoff occurs during summer and early fall storms under pre-development conditions due to high evapotranspiration and infiltration rates. Further, improved conveyance systems have less effect on the timing of peak streamflows for low intensity, long duration winter and spring storms than for high intensity, short duration summer and early fall storms.

The impacts of increased peak flow rates include increased risks to life and property. Stormwater management must minimize these risks.

3.5.1 Peak Flow Rate Criteria

Generally, accepted criteria are that maximum peak flow rates must not exceed pre-development values for storms with return periods ranging from 2 to 100 years. When measures to address water balance, erosion potential, and water quality are implemented, post-development runoff may be lower than pre-development runoff.

Peak flow rates must be determined on a site by site basis. Existing rates can be determined utilizing computer simulation modelling or by transposing a frequency analysis of measured peak flow rates on a unit area basis (Table 3.2) to a site. The latter approach will be more accurate. Computer simulation modelling will still be required to determine the impact of post-development attenuated runoff on peak flow rates at locations downstream of the site.

Table 3.5: Peak Flow Rates on a Unit Area Basis

Parameters	Humber River @ Elder Mills 02HC025	Rouge River near Markham 02HC022	Don River @ York Mills 02HC005
Drainage Area, km^2	303	186	88
2 Year Peak Flow Rate, m^3/s	35	43	25
2 Year Peak Unit Area, m^3/s/ha	0.0040	0.0049	0.0028
100 Year Peak Flow Rate, m^3/s	116	73	119
100 Year Peak Unit Area, m^3/s/ha	0.0132	0.0083	0.0135

Transposing a frequency analysis of measured flow rates should only be conducted for drainage areas upstream of the flow measurement location. Generally, unit area peak flow rates decline as drainage area increases. Transposition of measured flow rates upstream of the measurement location can be accomplished using computer simulation models.

3.5.2 Potential Impacts of Attenuated Runoff

Controlling post-development peak flow rates through storage to values less than pre-development conditions (overcontrol) may be required to maintain existing downstream watershed peak flow rates. Downstream rates can increase, although site runoff is controlled to pre-development levels. The timing of detained runoff peaks from specific points of a watershed may result in the coincidence of peaks. Providing site storage in the lower or mid portions of a basin will probably increase downstream peak flow rates as attenuated runoff will peak near the same time as upstream runoff. Controlling runoff in the upper portions may reduce downstream peak flow rates as the peaking times are significantly different. The potential impacts of site attenuated runoff on downstream watershed peaks should be calculated on a site by site basis.

3.6 Stormwater Management Practice Selection and Integration

As described in Chapter 2, a Subwatershed Study, and the more detailed Environmental Management Plan, provides a preferred environmental and stormwater management strategy including a series of stormwater management practices. Collectively, the practices included in the management strategy can achieve the environmental goals and objectives established for the Subwatershed.

The previous sections of Chapter 3 describe how objectives may be set for water balance maintenance, water quality protection, and control of erosion and flooding, if objectives have not already been established as part of the environmental planning process. The types of stormwater management controls suitable for addressing each issue have also been introduced (e.g., Table 1.3). Typically, a combination of stormwater management practices is required to meet the set of criteria addressing all water resource concerns. Lot level and conveyance controls, specifically the infiltration-based controls, are required to maintain the natural hydrologic cycle to the greatest extent possible. End-of-pipe facilities are usually required for flood and erosion control and water quality improvement, although lot level and conveyance controls can reduce the size of the end-of-pipe facilities required.

Alternative series of stormwater management practices, each meeting all of the established criteria, may be developed. It is worth re-emphasizing that the cumulative impacts of individual developments cannot be explicitly addressed without a Subwatershed Plan.

It should be confirmed that the proposed alternatives are feasible. Physical site constraints may preclude the use of certain stormwater management controls (Table 4.1). For example, native soils with low percolation rates may limit the use of infiltration type controls. There may be municipal standards or by-laws restricting the use of some SWMPs (e.g., reduced lot grading). Detailed design information for each SWMP, including possible constraints to use, is provided in Chapter 4. The integration of individual SWMPs in a Stormwater Management Plan that mitigates the multiple effects of urban development according to the established criteria is also described more fully in Chapter 4.

4.0 STORMWATER MANAGEMENT PLAN AND SWMP DESIGN

4.1 General

The stormwater management plan is the means by which water resource concerns are addressed during development. It will provide the size and location of SWMPs and will demonstrate (through modelling and other techniques) that when integrated, the SWMPs will meet the criteria established to ensure that:

- groundwater and baseflow characteristics are preserved;
- water quality will be protected;
- the watercourse will not undergo undesirable and costly geomorphic change;
- there will not be any increase in flood damage potential; and ultimately,
- that an appropriate diversity of aquatic life and opportunities for human uses will be maintained.

Ideally, planning for stormwater management will have begun early in the environmental planning process (Section 2.3) and will have been integrated with subdivision/site planning (Appendix A). The stormwater management plan and SWMP design can then incorporate opportunities which have been identified and designs which have been developed.

If a subwatershed plan has not been completed, the stormwater management plan will include much of the information that otherwise would be included in the larger scale plan.

The recommended strategy for stormwater management is to provide an integrated treatment train approach to water management that is premised on providing control at the lot level and in conveyance (to the extent feasible) followed by end-of-pipe controls. This combination of controls is the only means of meeting the multiple criteria for water balance, water quality, erosion control and water quantity.

Stormwater management strategies that employ a combination of SWMPs are desirable because they yield the following benefits:

- more effective stormwater management;
- reduction in land area required to implement end-of-pipe solutions;
- enhanced opportunities to integrate SWMPs effectively as amenities;
- decreased total cost when land value is factored in; and
- increased level of public awareness and involvement in the implementation and management of stormwater management initiatives.

To meet water quality objectives, a multi-component approach, in which there is a series of stormwater quality measures, may be used. If in a given situation no single measure is considered sufficient, then two or more controls in series may be expected to provide a higher level of water quality improvement. SWMPs used in a multi-component approach often include oil/grit separators, soakaway pits, sand and bioretention filters, vegetated filter strips, and grassed swales. If special concerns such as recreational activities involving water contact have been identified, additional water quality controls such as ultra violet disinfection units may be required.

Modelling techniques which may be used to develop the system of stormwater management practices are discussed in Section 4.9. The stormwater management plan will also include designs for the major and minor systems that convey the runoff from infrequent and frequent storm events, respectively (Section 4.8).

4.1.1 Lot Level and Conveyance Controls

Lot level and conveyance controls include those that are applied at the individual lot level, those which form part of the conveyance system, and controls which typically serve multiple lots but are only suitable for small drainage areas (< 2 hectares). They can be divided into two categories according to their primary function: storage controls and infiltration controls. Storage controls include:

- rooftop storage – restricting the discharge rate from roof drains to provide rooftop detention of stormwater
- parking lot storage – implementing catchbasin restrictors or orifices in the storm sewer to detain stormwater on parking lots
- superpipe storage – oversizing storm sewers and implementing orifices in the sewer to create pipe storage
- rear yard storage – implementing catchbasin restrictors in rear yard catchbasins to create rear yard storage

All of these measures were designed to detain stormwater to reduce peak runoff rates. In some cases there may be increased opportunity for evapotranspiration. However, detention times are typically short and these measures are not intended to reduce the volume of stormwater runoff. Further, they do not address water balance, erosion, or water quality issues. In addition, there are devices such as oil/grit separators which are suitable as lot level controls but which are also potentially suitable as end-of-pipe controls for small drainage areas with physical constraints. These devices are described in section 4.6.

Infiltration-based controls include:

- reduced grading to allow greater ponding of stormwater and natural infiltration;
- directing roof leaders to rear yard ponding areas, soakaway pits, or to cisterns or rain barrels;
- sump pumping foundation drains to rear yard ponding areas;
- infiltration trenches;

- grassed swales;
- pervious pipe systems;
- vegetated filter strips; and
- stream and valley corridor buffer strips.

The primary function of infiltration controls is to mitigate the impacts that urbanization normally has on the water balance (i.e., increased surface runoff, reduced soil moisture replenishment and groundwater recharge). Concentrated infiltration of stormwater collected from larger areas (e.g., infiltration basins, an end-of-pipe infiltration type control) will not match the characteristics of distributed infiltration which occurred under pre-development conditions. The natural hydrologic cycle can be maintained to the greatest extent possible by lot level infiltration controls.

Infiltration technologies can achieve water quality enhancement; however, stormwater containing high concentrations of suspended solids will tend to clog these controls. Further, infiltration of contaminated water can impair groundwater quality. Therefore, these measures are ideally suited to the infiltration of relatively clear stormwater, such as stormwater from rooftops which contains only atmospheric contaminants (i.e., contaminants deposited on the rooftop by precipitation or dryfall) or foundation drainage.

If the quality of the stormwater is such that there may be a problem with clogging in the system or degradation of groundwater quality, pre-treatment is required. Infiltration controls are not appropriate for applications with the potential for highly contaminated stormwater (e.g., industrial land uses).

By reducing the size of storm sewer infrastructure and end-of-pipe facilities, lot level and conveyance controls provide economic benefits. Section 4.8 provides guidance with respect to the reductions in end-of-pipe storage requirements which various lot level and conveyance controls allow.

The successful implementation of many lot level and conveyance measures require innovative subdivision design. In addition to the measures which are the focus of this manual, there are complementary controls which can be undertaken by home owners. For example, cisterns or rain barrels may be used in combination with bioretention gardens. Lot grading can be used to direct runoff to garden areas. Trickle irrigation systems may be used to make use of captured runoff in soils with lower infiltration capacities. Public education programs within municipalities can help to educate the public on the role they can play in the application of complementary measures.

A significant challenge in designing and implementing a stormwater management strategy which incorporates lot level techniques and other source controls is that many of these initiatives will be implemented on lands held in private ownership. Consequently, maintenance and the long-term effectiveness of the system is contingent on the actions of the landowner. Landowner education is the key to ensuring that systems remain effective over time. The successful application of lot level landscape solutions therefore requires the commitment of the municipality and the establishment of creative partnerships between the developer, municipality and landowner to realize consistent benefits over the long term.

4.1.2 End-of-Pipe Controls

End-of-pipe stormwater management facilities receive stormwater from a conveyance system (ditches, sewers) and discharge the treated water to the receiving waters. The purpose of end-of-pipe SWMPs is to control the impacts of urbanization which remain after lot level and conveyance controls have been applied. In most cases, new urban developments (unless they are small or of very low density) will require some sort of end-of-pipe SWMP. Some SWMPs that have been applied as end-of-pipe SWM facilities include:

- wet ponds;
- wetlands;
- dry ponds; and
- infiltration basins.

Other SWMPs applied in some cases as end-of-pipe facilities for smaller areas include filters and oil/grit separators.

Designs are possible which form a continuum from wetland to wet pond. The main criteria for differentiating between the types of facilities are the proportions of deep (> 0.5 m) and shallow areas (< 0.5 m). A wet pond has the greatest percentage of its volume provided in deep water zones. Aquatic plants are concentrated on shallow shelves around the perimeter. These shallow zones typically comprise less than 20% of the surface area of the facility. In contrast, wetlands are dominated by shallow zones (typically > 70% of the volume). Hybrid wet pond-wetland systems combine the two types in series with a minimum of 50% of the volume being provided in deep water areas.

Virtually all new wet facilities (i.e., those having a permanent pool) designed in Ontario have an extended detention storage component, in part due to the guidance provided in this document and its predecessor, but largely because of their multi-purpose design (i.e., water quality, quantity and erosion control). Extended detention storage refers to the active storage which is used during and after a runoff event, but which subsequently drains. The extent to which the active storage is filled by an event depends upon the volume of runoff. Section 3.3 provides a more detailed description of the functions of the permanent pool and active storage elements of ponds and wetlands.

Other jurisdictions draw a distinction between "wet ponds" and "extended detention wet ponds." In the former, water is discharged at the same rate as water is received, while in the latter, water is released more slowly.

Concerns are sometimes expressed regarding the ancillary aviary, terrestrial, and aquatic habitat provided by ponds and wetlands. While the design of these stormwater facilities does not specifically seek to create habitat conditions, habitat is usually created (as a natural consequence). Hence, there are often concerns relating to bio-accumulation of stormwater contaminants and the destruction of habitat during maintenance.

The loss of natural wetlands is, in itself, a major issue which has led to a provincial wetland policy. Some view stormwater wetlands as a replacement for lost natural wetlands; however, stormwater facilities will not have the same attributes as natural systems. It must be recognized that stormwater ponds and wetlands are first and foremost stormwater management facilities that must be maintained. They should not be considered as significant natural areas which require environmental protection. Equally important is that the use of natural wetlands for stormwater quality enhancement is not allowed since the introduction of stormwater may alter the hydrologic regime and chemical/biological composition of the wetland.

4.2 Siting of Stormwater Management Facilities

Site conditions must be addressed in the development of a stormwater management plan. Physical factors may suggest the use of particular SWMPs and preclude the use of others, or they may point to special design requirements (e.g., lining a pond). Factors which should be considered in the evaluation of the physical feasibility of SWMPs include:

- topography;
- soil type;
- depth to bedrock;
- depth to seasonally high water table; and
- drainage area.

Table 4.1 summarizes the physical constraints which could limit the use of lot level, conveyance, and end-of-pipe controls, and further detail is provided in the subsequent sections.

End-of-pipe SWMPs should normally be located outside of the floodplain (above the 100 year elevation). If the facility is multi-purpose in nature (e.g., providing quantity control in addition to quality and erosion control) it must be located above the highest design flood level. In some site-specific instances, SWMPs may be allowed in the floodplain if there is sufficient technical or economic justification and if they meet certain requirements:

- The cumulative effects resulting from changes in floodplain storage and balancing cut and fill do not adversely impact existing or future development;

- Effects on corridor requirements and functional valleyland values must be assessed. SWMPs would not be allowed in the floodplain if detrimental impacts could occur to the valleyland values or corridor processes;

- The SWMPs must not affect the fluvial processes in the floodplain; and
- The outlet invert elevation of the SWMP should be higher than the 2 year floodline and the overflow elevation must be above the 25 year floodline.

Table 4.1: Physical Constraints for SWMP Types

SWMP	Topography	Soils	Bedrock	Groundwater	Area
wet pond	none	none	none	none	> 5 ha
dry pond	none	none	none	none	> 5 ha
wetland	none	none	none	none	> 5 ha
infiltration basin	none	loam (min. inf. rate ≥ 60 mm/h)	> 1 m below bottom	> 1 m below bottom	< 5 ha
infiltration trench	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	< 2 ha
reduced lot grading	< 5%	loam (min. inf. rate ≥ 15 mm/h)	none	none	none
soakaway pit	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	< 0.5 ha
rear yard ponding	< 2%	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	< 0.5 ha
grassed swales	< 5%	none	none	none	< 2 ha
pervious pipes	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	none
vegetated filter strips	< 10%	none	none	> 0.5 m below bottom	< 2 ha
sand filters	none	none	none	> 0.5 m below bottom	< 5 ha
oil/grit separators	none	none	none	none	< 2 ha

In most cases, online facilities (those located within a watercourse) are discouraged because of concerns for wildlife movement, fish passage and disruption of energy inputs. On-line stormwater quantity facilities may be acceptable if designed such that the bank full flow, and hence fish movement, is not impeded/obstructed, and provided the foregoing requirements are met. On-line quality ponds can only be approved if issues of aquatic habitat can be resolved. An on-line facility can only be proposed in the context of a subwatershed plan.

In all development submissions, there should be a concerted effort to preserve the existing watercourse in its natural state. In cases where watercourse alterations are deemed necessary, the alterations should be designed using natural channel design techniques. This will require that the current stream reach be assessed using a stream classification system and that any alterations must account for hydrologic and hydraulic changes as a result of urbanization and the preferred stream function/morphology. Guidance on natural channel design techniques is provided in "Natural Channel Systems: An Approach to Management and Design" (Ministry of Natural Resources, 1994). Alterations to watercourses will, in most cases, require authorization under Section 35 (2) of the federal *Fisheries Act*.

The location of end-of-pipe stormwater management facilities is a contentious issue since the use of tableland reduces the overall area available for development. In an effort to minimize the loss of developable land, some municipalities allow the use of parkland dedication for SWMPs which offer recreational opportunities such as trails and playing fields. By offsetting the potential loss of land area available for development, the stormwater management facility, designed in whole or in part as usable parkland, may be considered acceptable.

4.3 Design Modifications for Cold Climates

An extensive review of cold climate design considerations was undertaken by the Centre for Watershed Protection (1997). Much of the information contained in the document is based on the results of a survey conducted among practitioners operating in cold climates. The major recommendations relating to cold climate design include:

- Increased storage volumes to account for volume reductions due to ice and effects of multi-day spring melt;
- Sizing and location of inlets and outlets to avoid ice clogging and freeze-up; and
- Prohibition of early spring drawdown for maintenance (avoid discharge of water with low oxygen or high chloride levels).

Many of the study's recommendations address depth of cover and backfilling practices which are standard in Ontario. However, the recommendations regarding storage volume increases and designs to limit problems due to freezing warrant consideration and are discussed further below.

4.3.1 Volume Modifications

A number of factors combine to make water quality treatment of winter runoff more difficult. Typically, a finer range of particles is washed off in winter due to the lower flow rates associated with melt events. Finer particles require longer to settle and so the relative removal efficiency may be expected to drop in winter. In many situations, ice formation will reduce the effective volume of a permanent pool (causing more frequent overflows or reduced event capture). Ice formation may eliminate a permanent pool entirely so that only active storage is available for treatment.

The Centre for Watershed Protection (CWP) design supplement suggests increasing the water quality volume provided in order to accommodate multi-day snowmelt events. This is not necessary for the volumes recommended in Table 3.2 of this manual because they were derived using a continuous modelling approach which accounts for snow accumulation and melt.

CWP also suggests adding compensatory volume to account for the expected maximum ice thickness and setting the minimum active storage volume to 25% of the total volume (active and permanent pool). The latter measure ensures that some treatment will continue to occur when the pool is completely frozen over. In most cases where water quality and erosion control storage are provided in the same facility, the minimum active storage volume for water quality will be met without the need for additional volume.

The former measure may be particularly important in more northern portions of the province where temperature regimes are such that ice cover may persist into the spring when runoff rates and contaminant washoff are more of a concern than they typically are during the winter. In these areas of the province, it is recommended that the permanent pool volume be increased by an amount equal to the expected volume of the ice cover.

The thickness of ice can be estimated using Stefan's Equation (Ashton, 1986):

$$h = \alpha(D_f)^{0.5}$$

Equation 4.1: Stefan's Equation

Where h = ice thickness in mm
α = coefficient of ice growth
D_f = the sum of freezing degree-days

Table 4.2 indicates values for the coefficient of ice growth. Work done at a pond in Kingston, Ontario, indicated that a coefficient value of 15 produced results close to measured values (Marsalek, 1997). The pond operated with a constant subsurface inflow, which tended to limit the build-up of ice. In general, it is expected that most ponds will be small enough and will receive sufficient inflow to behave more like a river (in terms of ice build-up) than a lake. Where possible, however, the designer should consult with the local municipality or conservation authority concerning local knowledge on ice depths.

Table 4.2: Coefficient of Ice Growth (α) (after Davar et al, 1996)

Condition	α (mm $°C^{-0.5}d^{-0.5}$)
Theoretical Maximum	34
Windy lakes with no snow	27
Average lake with snow	17-24
Average river with snow	14-17
Shelter river with rapid flow	7-14

4.3.2 Modifications to Inlet and Outlet Design

A number of the recommendations of the Centre for Watershed Protection (1997) study regarding inlets and outlets should also be considered:

- Slopes of inlet and outlet pipes should be > 1%;
- Minimum diameter of inlet and outlet pipes should be 450 mm;
- Submerged or partially submerged inlets should be avoided where possible; where a submerged inlet is required, its obvert should be located 150 mm below the expected maximum ice depth;
- Where a submerged berm is used to separate the forebay from the remainder of the pond, the crest of the berm should be set to ensure that there is adequate capacity to pass the design flow (accounting for the expected maximum ice depth);
- Submerged outlet configurations (reverse sloped pipe, baffle plate) should be set 150 mm below the expected maximum ice depth; reverse sloped pipes should have a minimum diameter of 150 mm; and
- Where a very small orifice (e.g., 75-100 mm) is required for discharge control, the design should provide for overflow caused by freezing (i.e., provide a secondary high-level orifice of larger diameter or other appropriate design feature.)

Specific recommendations for cold climate design are provided in the subsequent sections describing each SWMP.

4.4 Mitigation Measures for Increased Temperature

There are a number of reports which indicate that urban development end-of-pipe SWM facilities increase the temperature of water before it is discharged to the receiving waters (Beland, 1991, Galli, 1990, Schueler, 1992). Galli (1990) found that there is an increase in water temperature with all types of urban development SWMPs. These reports also stress, however, that an increase in water temperature is inevitable if an area is developed (i.e., urbanization causes stormwater temperature increases). This observation is based on current development practices. It is anticipated that employing the integrated approach to stormwater management described in Section 2.3 and Appendix A, and incorporating some of the techniques presented, will minimize the temperature increase associated with urbanization.

Literature values of temperature increases with different types of end-of-pipe stormwater management facilities have been recorded (Galli, 1990) and are provided in Table 4.3.

Table 4.3: Average Temperature Increases by SWMP Type*

SWMP Type	Temperature Increase
Infiltration Basin	1.4°C
Wetland (extended detention)	3.4°C**
Dry Pond (extended detention)	2.9°C
Wet Pond (extended detention)	5.1°C

* *Guideline based on monitoring – detailed calculations supersede this table.*

** *Original report value modified since the wetland design permanent pool was undersized resulting in a low ΔT.*

Wet ponds and wetlands can compound the temperature increase due to urbanization by maintaining water in the facility between storms and allowing it to acclimate to the air temperature. There are several techniques that can be used to reduce thermal impacts.

Pond Configuration

The configuration of a pond will affect its temperature. The length-to-width ratio should be maximized to prevent the occurrence of large open areas of water which cannot be shaded by riparian vegetation. Planted berms and islands can be used in ponds that have a poor configuration due to site specific topography/land availability.

Riparian Planting Strategy

Planting in the shoreline fringe and flood fringe zones of a wet pond help to shade the pond and minimize temperature increases during inter-event periods. The planting strategy should incorporate designs which shade open water areas when the vegetation reaches maturity.

Bottom-draw Outlet

There are temperature benefits from a bottom-draw facility, although this is dependent on the size of the permanent pool and the release depth. There is a minimal difference in water temperature within the top metre of a permanent pool. Lower temperatures (in the order of several degrees Celsius) occur several metres below the permanent pool surface. Ponds with permanent pool depths greater than 3 metres, however, are likely to become thermally stratified during the summer months. The water at depth can become anoxic, and there is a potential for metals and nutrients to be remobilized. Although the oxygen deprivation can be solved by re-aeration at the outlet (e.g., discharge over rocks), the discharge of the polluted water would be undesirable. Accordingly, ponds with a very deep release (> 3 m) should consider re-aeration in the pond itself to prevent thermal stratification from occurring. However, this may reduce sedimentation and resuspend sediment collected at the pond bottom.

Subsurface Trench Outlet

Treatment of water, by routing the discharge through a subsurface trench filled with clear stone, has also been suggested to reduce temperature. As the water flows through the trench, heat is transferred to the stone. It is purely a conveyance system which does not rely on infiltration; however, there is relatively little knowledge with respect to the success of these systems.

The dimensions of the system depend on the intended range of release rates, and the proximity of the pond to the watercourse. The length of the trench should be maximized to increase the opportunity for heat transfer. The cross-sectional area of the trench should be sized based on the design conveyance flow which does not necessarily have to match the design release rate from the pond (especially if the pond will accommodate the runoff from relatively large storms; i.e., > 25 mm). The trench should be designed to accommodate frequent events (i.e., ≤ 10 mm) which will have a greater effect on the thermal regime of the receiving water.

The trench should be wrapped with non-woven filter fabric to prevent the native material from blocking the pore space in the stone/rock. The stone should be relatively small (13 mm - 25 mm) since smaller stones will have a greater total surface area available for heat transfer.

Night Time Release

Monitoring evidence (Beland, 1990 unpublished) suggests that the water in stormwater ponds cool during the night as a result of ambient temperature fluctuations that can be up to 5°C. Generally, the lowest pond temperatures were recorded during the early morning (5 A.M. - 7 A.M.) indicating that very early morning releases should be targeted for facilities which are designed with real time controls.

Outlet Channel Design

In cases where there is a lengthy outlet channel from the end-of-pipe SWM facility to the receiving waters, natural channel design techniques can be employed. Guidance on natural channel design techniques is provided in "Natural Channel Systems: An Approach to Channel

Management and Design" (Ministry of Natural Resources, 1994). The outlet channel from an end-of-pipe SWM facility to the receiving waters should be shaded by plantings to minimize the temperature of the water discharged to the receiver.

4.5 Lot Level and Conveyance Controls

4.5.1 Rooftop Storage

Flat building roofs can be used to store runoff to reduce peak flow rates to storm sewer systems. Rooftop storage is economical and requires little extra cost during construction. It is generally applicable to large flat commercial and industrial rooftops. Residential roofs are usually peaked with few opportunities for storage. Rooftop storage is widely applied for infill development scenarios to mitigate the need for downstream storm sewer size increases. Few regulatory agencies allow rooftop storage to reduce downstream post-development peak flow rates for new sites. Local regulatory agencies should be consulted to determine what is allowable.

Design Guidance

Storage Volumes and Release Rates

Plates or weirs are used in rooftop drainage hoppers to control the rate of discharge and depth of storage. Drainage hoppers can discharge to the internal building drainage system or externally to the ground.

Discharge rates for pre-manufactured rooftop drainage hoppers are specified by the manufacturer. Typically, discharge values for each hopper can range from 1 to 15 L/s. Storage is user determined for dead level or slightly sloped roofs. Large commercial roofs can store 50 mm to 80 mm of runoff. The 100 year 24 hour rainfall amount for southern Ontario is approximately 100 mm. Detention times are usually between 12 and 24 hours. Structural/mechanical engineers should supervise the detailed design of rooftop storage to ensure that loadings are not exceeded.

Emergency Overflows

Openings must be designed in the roof parapet walls to ensure loading requirements are not exceeded during excessive runoff events or in case roof hoppers are blocked. Openings will limit the depth of ponding and the loading to the building. Rooftop storage requires periodic inspections and prompt maintenance. More frequent inspections for blockages should be made during the winter and fall months.

Hopper Location

The location of hoppers is a function of the building roof length, width and slope. The manufacturer commonly supplies guidelines for determining the location of the hoppers. Typically, hoppers are located 10 m to 15 m from the roof parapet and the distance between hoppers is approximately 30 m.

Drain Pipe Sizing
Vertical drain pipes should be sized to convey the maximum flows discharged from the roof hoppers. Vertical pipe capacities range from 2 L/s for 50 mm diameter pipes to 40 L/s for 150 mm diameter pipes.

Leakage
Improved waterproofing techniques are required during construction to prevent leakage of ponded runoff.

Technical Effectiveness
Rooftop storage is highly effective in reducing downstream peak flow rates. However, the volume of stormwater runoff to the sewer system is not reduced.

4.5.2 Parking Lot Storage

Parking lots can be used to store runoff to reduce peak flow rates in storm sewer systems. Parking lot storage is economical with slightly increased costs for construction. It is generally applicable to commercial and industrial lots. Parking lot storage is not used in residential areas due to the small parking areas. It has been widely applied for infill developments to mitigate the need for downstream storm sewer size increases. Few regulatory agencies allow parking lot storage to reduce downstream post-development peak flow rates for new sites. Local regulatory agencies should be consulted to determine what is allowable.

Design Guidance
Storage Volumes and Release Rates
Storage is created when runoff rates are greater than the outflow allowed by the restricted capacity of inlet control devices (ICDs) placed in maintenance holes or catchbasins. Pre-manufactured ICDs can take the form of an orifice plate or plug over the pipe to a catchbasin or maintenance hole. Specialty ICDs can be orifice plates under catchbasin grates.

Generally, ponding depths are limited to 300 mm with durations less than 1 hour. Design criteria can vary significantly between municipalities. Some municipalities require sizing of the control to the pre-development 2 year peak runoff rate for all (2 through 100 year) post-development runoff rates.

Pre-manufactured ICDs have patentable dimensions and discharge rating curves. Simple spreadsheet calculations can be used to size parking lot storage, or computer simulation programs can be used. Calculations may be complicated when building roof storage discharges to surface ponding areas or when storm sewers are used to control ponding levels at catchbasins.

Parking Lot Grading
Storage at a catchbasin or manhole is limited by the maximum ponding depth and the grade of the parking lot. Generally, grades are greater than 0.5% and less than 5%. Storage capacities are reduced for increasing grades assuming a constant ponding depth.

Emergency Overflows

Overland flow paths must be designed to accommodate runoff that exceeds the storage capacity at the catchbasins. Debris blocking at the catchbasin grate can reduce outflow rates and create overflows. Overland flow paths can be sewers, swales or the roadway system.

Control Location

ICDs can be located in catchbasins or at maintenance holes located on the property boundary. Controls at the property boundary allow the municipality to check the operation of the control at its convenience. Controls at the property boundary will likely result in constant water levels at all catchbasins within the site. Controls at the catchbasins allow for different ponding levels and maximize storage on steeply graded sites.

Ponding Locations

Frequent ponding areas should be located away from buildings within the site. Storage of runoff during an event can be a nuisance to parking lot users.

Technical Effectiveness

Parking lot storage is highly effective in reducing downstream peak flow rates; however, the volume of storm runoff to the sewer system is not reduced. Normal parking lot maintenance procedures are suitable for parking lot storage areas.

4.5.3 Superpipe Storage

A superpipe, consisting of pre-manufactured pipe requiring on-site assembly, can reduce peak flow rates by providing subsurface storage. There are marginal water quality benefits as some of the coarse sediment may settle. Generally, superpipes are utilized for small development sites which lack sufficient surface space to construct detention facilities. Design and construction standards for superpipes are defined by the local municipality or township.

Superpipes are equipped with small outlet pipes. As inflow rates are much larger than outflow rates, runoff is detained. Generally, detention times are on the order of a few hours. Compared to traditional surface facilities, land requirements for superpipes are small while material costs are high.

Design Guidance

Inlets and Outlets

Inlets and outlets must be sized for each site. Outflow rates may have been previously defined in master drainage plans or storm sewer analyses. Generally, inlets are the on-site storm sewer system that has been designed to convey frequent runoff events from impermeable surfaces. Outlets are much smaller pipes that may discharge to watercourses or existing storm sewer systems.

Length and Diameter

The length and diameter of the superpipe will be a function of the storage required to meet off-site discharge rates. Maximum pre-manufactured diameters are approximately 3 m. However, the maximum pipe size that can be transported to the site due to constraints, such as bridge height, must be considered. Some municipalities have established minimum sizes to facilitate cleaning. Generally, pipes with diameters less than 1.8 m are difficult to clean.

Slope

Minimum slopes are approximately 0.5% as a slope must be maintained to completely drain the pipe. Slopes should be kept to the minimum as steep slopes will reduce the amount of storage available within the pipe.

Emergency Overflows

Emergency surface overflow paths should be located and sized to convey the 100 year runoff in case the superpipe (inlet/outlet) becomes plugged or inoperable.

Location

Superpipes should be installed where the pipes can be easily excavated for maintenance. Suitable locations include parking lots and grassed swales adjacent to property boundaries. Superpipes should be located in close proximity to fire hydrants that supply flushing water for the removal of sediments.

Maintenance

Personnel access points should be located at the upstream and downstream ends of the superpipe. For safety precautions, confined space entry procedures will be required for maintenance personnel when flushing or removing sediments. Upstream removal of sediments will extend the maintenance interval (oil/grit separators and/or other pre-treatment devices can be used for this purpose).

Technical Effectiveness

Superpipes are very effective in reducing site peak flow rates; however, they do not significantly improve water quality. Special maintenance considerations are required for clearing.

4.5.4 Reduced Lot Grading

Typical development standards require minimum lot grades of 2% for adequate drainage of stormwater away from a building. Alternative Development Standards (Ministry of Municipal Affairs and Housing, 1995) suggest reducing minimum lot grades from 2% to 0.5%. Despite this, the designer should check the acceptability of this practice with the local municipality.

Design Guidance

Soils

Reduced lot grading can be implemented where soils have a percolation rate ≥ 15 mm/h. This generally includes all soils coarser than a loam (Table 4.4). Table 4.4 should be used as a screening tool to determine if site conditions may be suitable for infiltration type controls. However, site testing should be undertaken to confirm actual soil percolation rates.

Scarification, or tilling of the soil to a depth of approximately 300 mm, will enhance infiltration; thereby helping to overcome the soil compaction that normally occurs during construction.

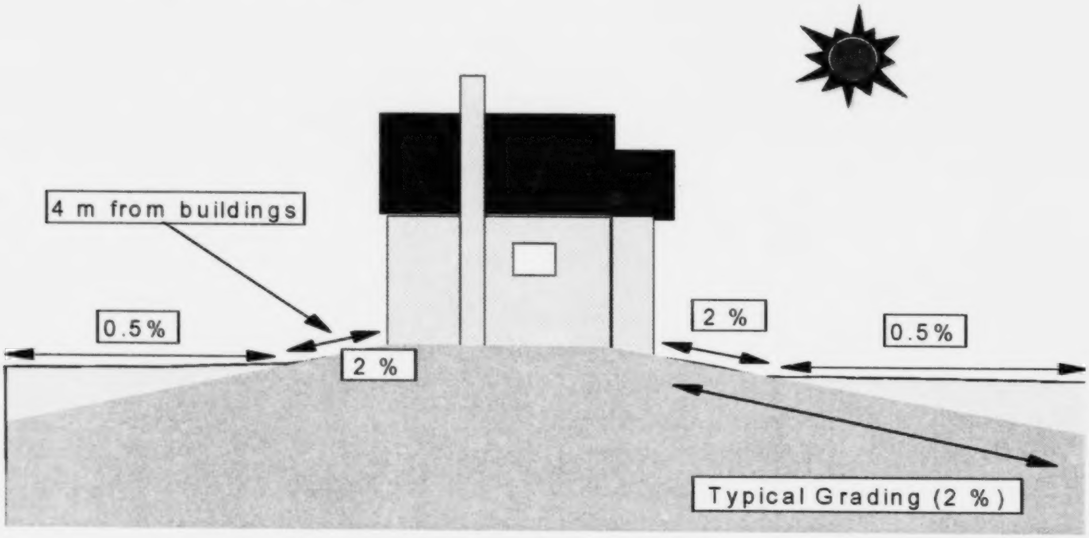
Table 4.4: Minimum Soil Percolation Rates

Soil Type	Percolation Rate (mm/h)
sand	210
loamy sand	60
sandy loam	25
loam	15

Topography
A reduction in the lot grading should be evaluated if the land is naturally flat. In hilly areas, alterations to the natural topography should be minimized (as indicated in Appendix A).

Setbacks
In order to ensure that foundation drainage problems do not occur, the grading within 2 metres - 4 metres of a building should be maintained at 2% or higher (local municipal standards should be reviewed to ensure that the grading around a building is in compliance). Areas outside of this boundary may be graded at less than 2% to create greater depression storage, and promote natural infiltration (Figure 4.1).

Figure 4.1 Lot Grading Changes



Roof Leaders

In areas where reduced lot grading is implemented, roof leaders which discharge to the surface should extend 2 metres away from the building.

End-of-Pipe Benefits

End-of-pipe extended detention requirements will be reduced by decreasing lot grades. The benefits of reduced grades can be assessed based on an increase in pervious area depression storage in hydrological modelling. It is recommended that the pervious depression storage (or initial abstraction) be increased by 1.5 mm for a change in lot grades from 2% to 0.5% (based on a typical lot of 12 m × 30 m). Section 4.8 provides further guidance with respect to adjusting end-of-pipe SWM facility storage to account for lot level and conveyance controls.

Technical Effectiveness

There is little experience with reduced lot grading as a standard practice on a subdivision scale. The largest impact this practice will have is on the homeowner's utility of his or her land. The water ponded on lots may take 24 to 48 hours to drain which may restrict the active use of the land. This impact will be greatest in spring, but negligible during the summer. It is anticipated that the public will be receptive to this alternative standard if they understand its benefits.

4.5.5 Roof Leader to Ponding Areas

An area for ponding can be created in the rear yard or along the rear lot line. Roof leaders are discharged to the surface and directed to the ponding area. Water is detained in the ponding area until it either evaporates or infiltrates.

A variation on this SWMP adds rainbarrels or cisterns, so that water can be stored for later use in the garden or on the lawn. This is particularly useful in areas with impermeable soils, where infiltration is slow and ponding areas may remain wet for an extended period of time.

Design Guidance

Soils

Ponding can be implemented for soils having a percolation rate $\geq$ 15 mm/h. Infiltration can be improved by tilling the ponding area to a depth of approximately 300 mm before sod is laid.

Storage Volume

A minimum storage volume of 5 mm over the rooftop area should be accommodated in the rear yard without overflowing. The maximum target storage volume should be 20 mm over the rooftop area since 90% of all daily rainfall depths are less than this amount.

Ponding Depth

The area for ponding should be a shallow depression with a maximum depth of 100 mm. An overland flow path should be established for depths greater than this amount.

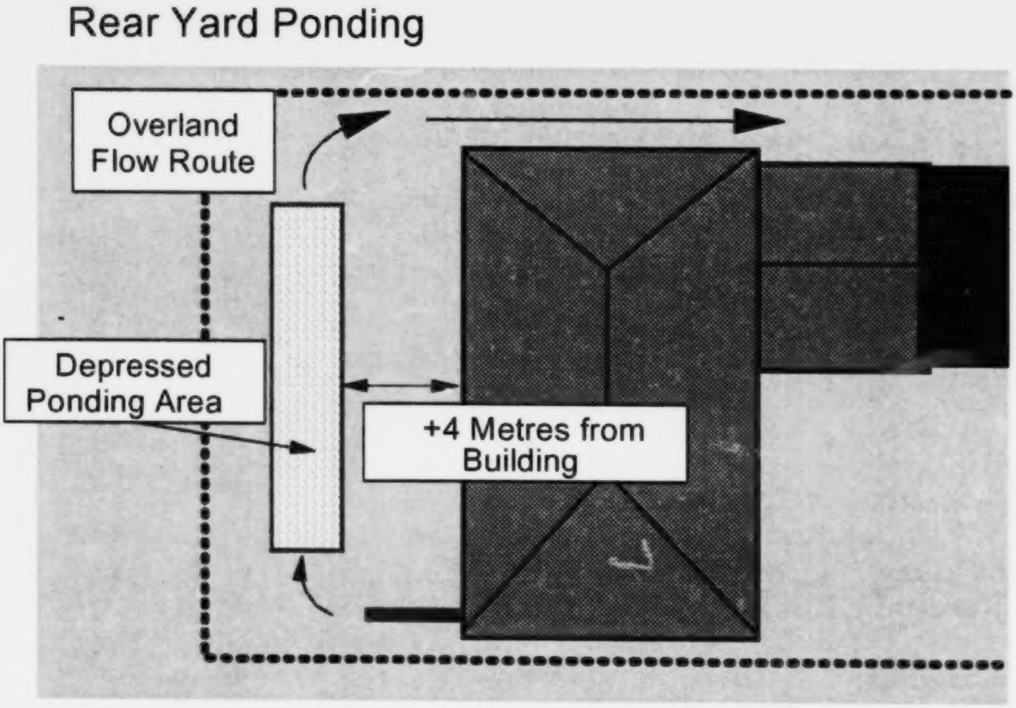
Configuration

The configuration of the ponding area will depend on the site-specific layout of the development. The depth of ponding should be minimized as a goal. If possible the length of ponding should be maximized compared to the width to prevent short-circuiting to reduce the potential for groundwater mounding, and to maximize the potential for infiltration.

Location

The area of ponding should be at least 4 metres away from any building foundations to ensure that the ponded water does not increase the amount of foundation drainage (Figure 4.2).

Figure 4.2: Roof Leader Discharge to Rear Yard Areas



In general, surface ponding areas should not be located over sewage system leaching beds to minimize the potential for compaction of the leaching bed and groundwater mounding problems. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field in areas where surface ponding is proposed.

In general, surface ponding areas should not be located adjacent to highways or major roads.

Common Ponding Areas

Ponding areas can be created along the rear lot lines by raising rear yard catch-basins such that they are used as an overflow system. Infiltration in the ponding areas can be enhanced by providing an infiltration trench system underneath the swale (see Sections 4.5.6 and 4.5.8).

Roof Leader

The roof leader should discharge into the ponding area via a splash pad and overland flow route.

End-of-Pipe Benefits

An increase in impervious depression storage is the simplest way to assess the benefits of rear yard storage on the end-of-pipe extended detention requirements. If modelling is performed, this would require the rooftop area to be modelled separately from the other areas. The increased depression storage would be equivalent to the depth of storage provided over the entire rooftop area. Further guidance is provided in Section 4.8.

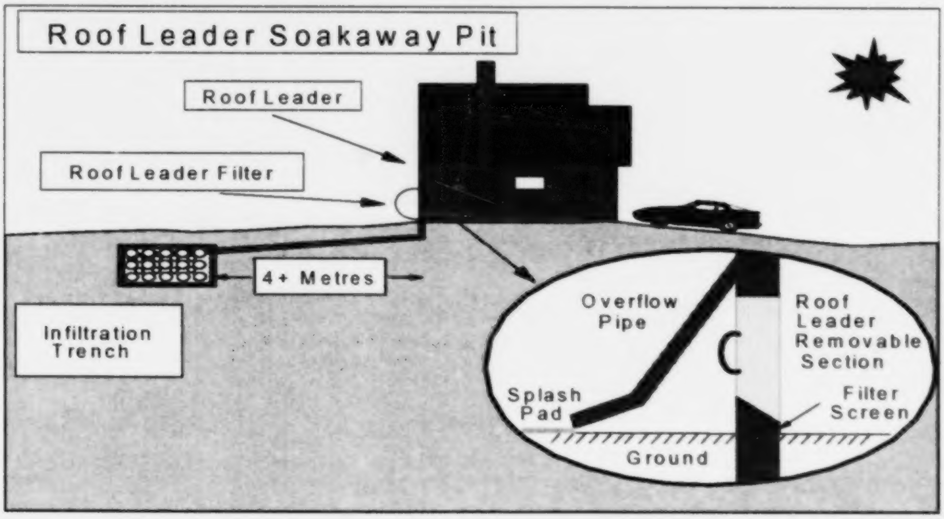
Technical Effectiveness

This technique has similar benefits and drawbacks to flatter lot grading. The benefits of this practice outweigh its drawbacks, and therefore, this stormwater lot level control is recommended.

4.5.6 Roof Leader Discharge to Soakaway Pits

This stormwater lot level control infiltrates roof drainage via an underground infiltration trench (soakaway pit). The roof drainage is conveyed directly to the trench by the roof leader (Figure 4.3). The term soakaway pit is typically used to describe an infiltration trench which serves a single lot and which does not receive road runoff. Since the runoff is relatively clear, pre-treatment is not needed.

Figure 4.3: Roof Leader Discharge to Soakaway Pit



Design Guidance

Water Table Depth

The depth from the bottom of the soakaway pit to the estimated seasonally high water table should be greater than or equal to 1 metre.

Depth to Bedrock

The depth from the bottom of the soakaway pit to the bedrock should be greater than or equal to 1 metre.

Soils

Soakaway pits can be used where soils have a percolation rate ≥ 15 mm/h. This generally includes all soils coarser than a loam.

Storage Volume

A minimum storage volume of 5 mm over the rooftop area should be accommodated in the soakaway pit without overflowing. The maximum target storage volume should be 20 mm over the rooftop area since 90% of all daily rainfall depths are less than this amount.

Storage Configuration

The length and width of a soakaway pit are dependent on the configuration of the development. The length of trench (in the direction of inflow) should be maximized compared to the width to ensure the proper distribution of water into the entire trench and to minimize the potential for groundwater mounding.

The permeability of the native soil will dictate the maximum allowable underground storage depth as indicated by Equation 4.2. Storage depths greater than 1.5 m are generally not recommended for soakaway pits from both a cost, and a compaction perspective. The weight of the water in a deep soakaway pit will compact the surrounding native soil and decrease the infiltration capacity.

There are exceptions, however, to this maximum depth recommendation. In areas with deep sand lenses or significant horizontal soil stratification, deep soakaway pits may be preferred. If, for example, a sand lens is located at a depth of 2 metres, it would be advantageous to construct a deep soakaway pit which drains into the lens. Soils investigations should be undertaken to determine whether these conditions exist.

$$d = \frac{PT}{1{,}000}$$

Equation 4.2: Maximum Allowable Soakaway Pit Depth

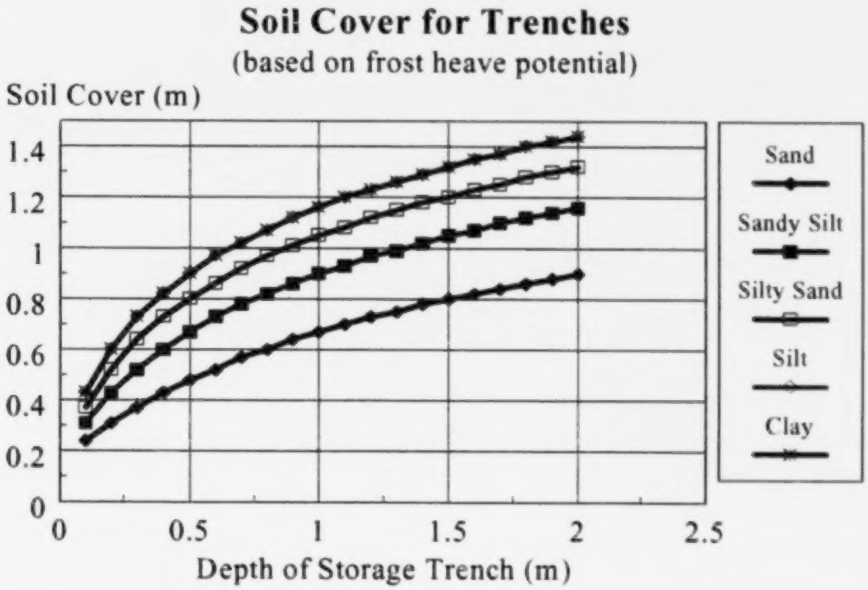
where d = maximum allowable depth of the soakaway pit (m)
P = percolation rate (Table 4.1) (mm/h)
T = drawdown time (24 - 48 h) (h)

It is recommended that a conservative drawdown time (24 h) be chosen recognizing that the percolation rates into the surrounding soil will decrease over time and that there will likely be a lack of maintenance in some cases.

Soil Cover

Typically, the pit should be located close to the ground surface; however, this will depend on the depth of storage in the trench, the potential for frost heave, and the stratification of the surrounding soil. The potential for frost heave is dependent on the native soils and the volume of water in the trench which may freeze. Figure 4.4 provides guidance on the recommended minimum soil cover for various subsurface trench depths and native soils. It is based on professional opinion, the expansion of water because of freezing, and the potential availability of water to freeze. Ice lens formation is not anticipated to occur within the trench because of the size of the pores in the storage media.

Figure 4.4: Soil Cover for Trenches



Location

The roof leader is extended underground to an excavated infiltration trench. The trench should be located at least 4 metres away from the foundation of the nearest building to prevent excessive foundation drainage.

Groundwater mounding calculations may be required to ensure that soakaway pits do not interfere with sewage system leaching beds. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field.

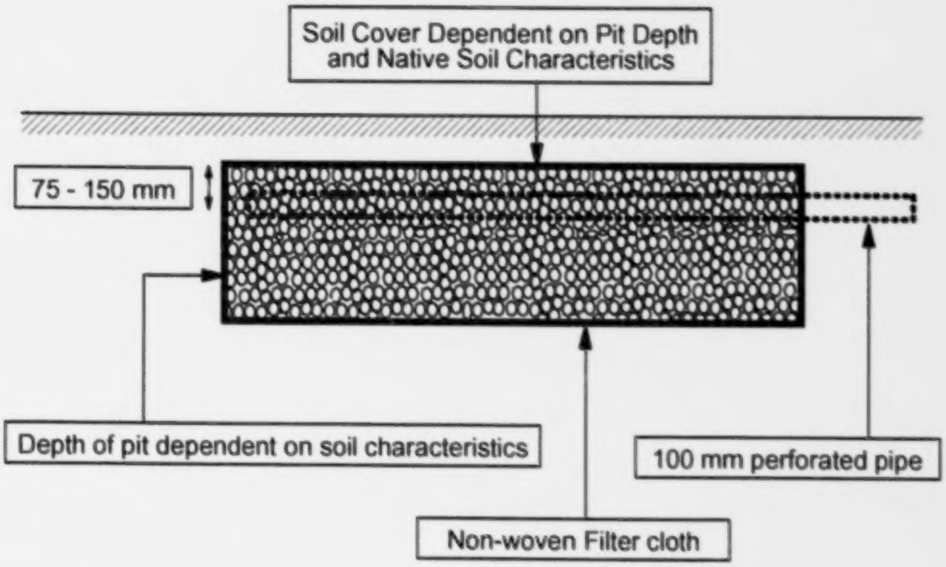
It is anticipated that calculations will be required in areas where the soils are marginally acceptable for infiltration.

Common Soakaway Pits
Common soakaway pits may be viable in areas with compact built forms. The common soakaway pit can be located in neighbourhood park areas or along rear lot lines.

Storage Media
The trench is comprised of clear stone (50 mm diameter). Non-woven filter cloth should be used to line the trench to prevent the pore space between the stones from being blocked by the surrounding native material.

Conveyance Pipe
The roof leader should extend into the soakaway pit for the full length of the pit. The extension of the roof leader should be perforated to allow water to fill the pit along the length of the pipe. The perforated pipe should be located near the surface of the trench (75 mm - 150 mm from the top of the pit). A typical trench detail is shown in Figure 4.5.

Figure 4.5: Soakaway Pit Details


Soil Cover Dependent on Pit Depth and Native Soil Characteristics
75 - 150 mm
Depth of pit dependent on soil characteristics
100 mm perforated pipe
Non-woven Filter cloth

Overflow By-pass
An overflow pipe should be installed from the roof leader that discharges to a splash pad. A removable filter should be incorporated into the roof leader below the overflow pipe. The filter should have a screened bottom to prevent leaves and debris from entering the soakaway pit. It

should be easy to remove so that a homeowner can clear the filter. Frequent use of the overflow pipe will indicate the need for filter screen maintenance.

Technical Effectiveness

Soakaway pits for roof leader drainage have been implemented in numerous areas (e.g., Toronto, Maryland). Lindsey et al. (1992) indicated that of 25 soakaway pits monitored, 60% were operating as designed.

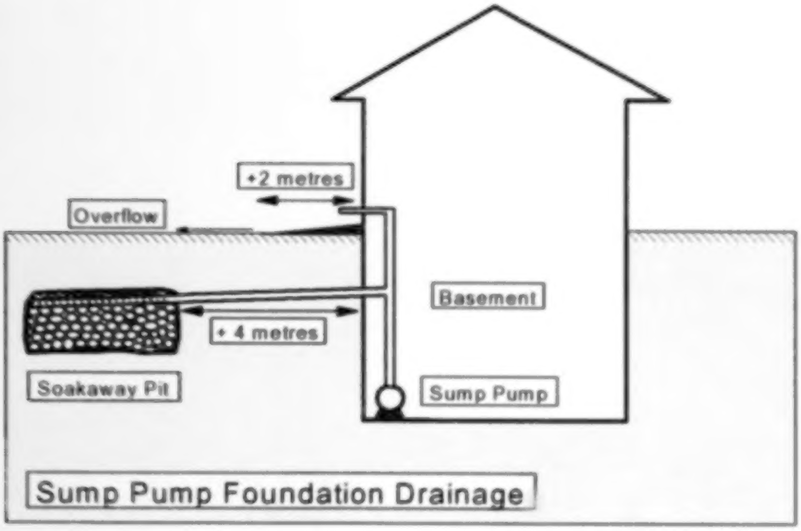
Soakaway pits have both benefits and drawbacks compared to rear yard ponding. The benefits include greater recharge (less evapotranspiration) and less inconvenience to the homeowner (less surface water ponding). The drawbacks include greater maintenance and uncertain longevity.

For soakaway pits accepting only roof drainage, the potential for clogging is low as is the risk of groundwater quality degradation. Accordingly, this SWMP is recommended for general implementation.

4.5.7 Sump Pumping of Foundation Drains

Development standards allow foundation drains to be connected to the storm sewer. Alternative standards (Ministry of Municipal Affairs and Housing, 1995) allow the use of sump pumps to discharge foundation drainage to the surface or soakaway pits (Figure 4.6). Because foundation drainage is relatively clear water, the cost of stormwater management and sewage treatment can be reduced by keeping it separate from storm and sanitary sewers. The municipality should be contacted before recommending this type of control as its use may not be permitted.

Figure 4.6: Foundation Drainage Options



Design Guidance

Water Table Depth

In areas where the seasonally high water table is within 1 metre of the building foundation drains, sump pumps should not be utilized. This requirement is imposed to prevent excessive sump pump operation and to prevent a looped system whereby the sump pump discharges maintain the foundation drainage. Where the use of sump pumps is not feasible, a "third pipe" may be used to convey foundation drainage to the receiving water.

Depth to Bedrock

In areas where the depth to bedrock is within 1 m of the foundation drain elevation, foundation drainage by sump pumps is not feasible. This requirement is imposed to prevent excessive sump pump operation and a looped system.

Location

Figure 4.6 demonstrates foundation drainage to a soakaway pit using a sump pump. If a soakaway pit is used, it should be located a minimum of 4 metres away from all building foundations to minimize the contribution of soakaway pit drainage to foundation drainage. If the foundation drains are being discharged directly to the surface, the discharge point at the ground surface should be located at least 2 metres away from all building foundations, and there should be sufficient grade from the foundation wall away from the building ($\geq 2\%$) for 2 metres to 4 metres to convey the foundation drainage away.

Overland Flow

Discharges to the surface should be directed to the rear yard to minimize the amount of surface drainage over sidewalks during the winter. Sump pumps discharging to the surface should discharge approximately 0.5 m above the ground surface to prevent blockages in the winter due to ice and snow.

4.5.8 Infiltration Trenches

Infiltration trenches in this manual refer to infiltration systems with a subsurface storage component that treat stormwater runoff from several lots as opposed to soakaway pits which are primarily used for a single lot application. Infiltration trenches can be implemented at the ground surface to intercept overland flows, or underground as part of a storm sewer system.

The acceptability of infiltration trenches should be confirmed because of potential concerns for aquifer contamination. In most cases, infiltration trenches will provide marginal flooding and erosion control benefits because they are sized for recharge and water quality.

Design Guidance

Drainage Area

Infiltration trenches can be implemented for small drainage areas (< 2 ha). The use of trenches for larger drainage areas is inappropriate due to the problems associated with infiltrating a large volume of water in a relatively small area of land. Groundwater mounding problems, compaction, and sealing of the native soil material have a higher potential of occurrence as the volume of water to be infiltrated increases. Further, in terms of the natural hydrological cycle, concentrated infiltration of stormwater collected from large areas will not match the characteristics of pre-development distributed infiltration.

Land Use
Infiltration trenches can be implemented for residential land uses. Trenches are best implemented for compact housing (cluster housing, townhouses) in small parks/greenspace areas where several households can drain to a single trench.

Infiltration trenches are not suitable for industrial land uses since there is a high potential for groundwater contamination and/or dry weather spills. Similarly, infiltration trenches are not suitable for commercial parking lots since there is a high potential for dry weather spills and for chloride to enter the trench, and subsequently, the groundwater system.

Water Table Depth
The seasonally high water table depth should be > 1 m below the bottom of the infiltration trench.

Bedrock Depth
The depth to bedrock should be > 1 m below the bottom of the infiltration trench.

Soils
Infiltration trenches are not suitable if the native soil has a percolation rate less than 15 mm/h.

Storage Configuration
The depth of the storage layer should be sized to ensure a 24 to 48 hour drawdown (a 24 hour drawdown is recommended) of the stored water based on the percolation rate determined in the field. Equation 4.2 can be used to calculate the maximum allowable storage depth in the trench.

A maximum storage volume equal to the runoff from a 4 hour 15 mm storm should be provided in the trench storage media if the trench accepts runoff from several lots. The length and width of the trench will be determined by the characteristics of the site in question (topography, size and shape).

If a surface trench is designed, the dimensions of the trench will depend on the path of influent water. If stormwater is conveyed to the trench as uniform sheet flow, the length of the trench perpendicular to the flow direction should be maximized. If stormwater is conveyed as channel flow, the length of trench parallel to the direction of flow should be maximized.

In a subsurface trench, the water is conveyed into the trench via a pipe system. In this arrangement, it is recommended that the trench length (parallel to the incoming pipe) be maximized compared to the trench width. This will encourage the uniform distribution of water in the storage layer.

The appropriate bottom area of the trench can be calculated using Equation 4.3. This equation assumes that all of the infiltration occurs through the bottom of the trench.

Equation 4.3: Infiltration Trench Bottom Area

$$A = \frac{1{,}000V}{Pn\Delta t}$$

where A = bottom area of the trench (m^2)
V = runoff volume to be infiltrated (Table 3.2)
P = percolation rate of surrounding native soil (mm/h)
n = porosity of the storage media (0.4 for clear stone)
Δt = retention time (24 to 48 hours)

Location/Setbacks

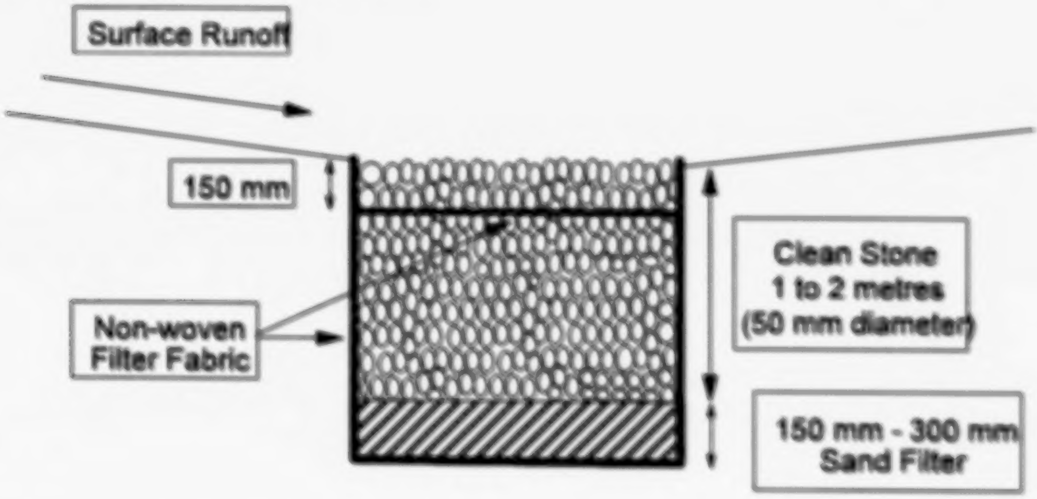
Groundwater mounding calculations may be required to ensure that infiltration trenches do not interfere with sewage system leaching beds. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field. It is anticipated that calculations will be required in areas where the soils are marginally acceptable for infiltration.

The setbacks from wells specified in the Building Code for leaching bed systems shall also be observed for infiltration trenches.

Storage Media

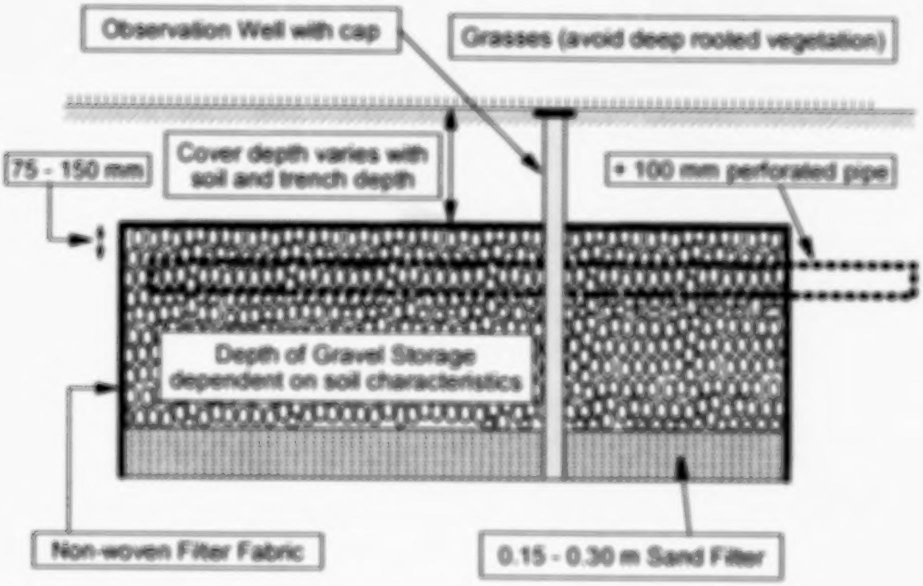
The storage media holds the stormwater until it can percolate into the surrounding native material. It is recommended that 50 mm diameter clear stone be used (Figure 4.7). While gravel is the most common medium used, precast infiltration storage media are available which are also generally acceptable.

Figure 4.7: Surface Infiltration Trench



Non-woven filter fabric should be installed at the interface of the trench and the native material to ensure that the latter does not clog the trench. For subsurface trenches, the filter fabric should extend to cover the top of the trench (Figure 4.8). However, this may need to be frequently replaced depending on the volume of suspended solids transported to the trench.

Figure 4.8: Subsurface Infiltration Trench


Observation Well with cap
Grasses (avoid deep rooted vegetation)
75 - 150 mm
Cover depth varies with soil and trench depth
• 100 mm perforated pipe
Depth of Gravel Storage dependent on soil characteristics
Non-woven Filter Fabric
0.15 - 0.30 m Sand Filter

Filter Layer

A filter layer is constructed underneath the storage layer to provide quality enhancement of the stormwater before it infiltrates the native soil. The most common filter medium used in infiltration trenches is sand. The sand layer should be approximately 0.3 m thick (0.15 m - 0.30 m). Grain size is specified based on the effective size (d_{10} – 10% of the particles are less than this size) and coefficient of uniformity (C_u – the larger the number, the less uniform the material). Sands with an effective size of 0.25 mm and $C_u \leq 3.5$, or an effective size of 2.5 mm with a $C_u \leq 1.5$, are recommended for filter material.

Peat may be mixed with the sand to enhance the pollutant removal characteristics of the trench. Peat has a high affinity for metals, hydrocarbons, and nutrients (Galli, 1990). Fibric or hemic peat should be utilized to achieve the desired percolation rates. Sapric or well-decomposed peat is discouraged since this type of peat has a slow percolation rate. The percolation rate of filter media should equal or exceed the percolation rate of the native soil material.

Planting Strategy

The planting requirements for an infiltration trench are more aesthetic than functional. Grass/herb mixtures are generally acceptable. Plantings with deep roots should be avoided since they can puncture the filter fabric at the top of the trench allowing native soil material to clog the gravel storage layer from above.

Distribution Pipes

In an underground trench, water is conveyed into the storage layer by a series of perforated pipes. A header pipe connects to the influent storm sewer and distributes the flow into lateral perforated pipes (≥ 100 mm diameter) which traverse the entire length of the trench. The lateral pipes should be spaced a maximum of 1.2 m apart. The perforated pipes should be located approximately 75 mm - 150 mm from the top of the storage layer.

Overflow/By-pass Pipe

A by-pass pipe should be incorporated into the design of an underground infiltration trench to convey high flows around the trench. This will necessitate the construction of a flow splitter upstream of the trench (see Section 4.7). The by-pass pipe may also function as the:

- normal outlet until the site is stabilized (inlet pipe to trench blocked off);
- normal outlet during trench maintenance; and
- normal outlet during winter/spring conditions.

Pre-treatment

In residential areas, where the infiltration trench takes primarily roof and pervious area runoff, the technique can be employed without pre-treatment. If the infiltration trench is being used to treat stormwater runoff from an entire site (including roads and parking lots), pre-treatment is necessary to minimize the potential for suspended sediments to clog the trench.

Sand filters, vegetated filter strips, grassed swales and/or oil/grit separators may be used. Pollution prevention through source controls should also be investigated (sanding/salting practices, public education with respect to street/driveway sediments) in areas where an infiltration trench is proposed.

Construction

Infiltration trenches will only operate as designed if they are constructed properly. There are three main rules that must be followed during the construction of an infiltration trench:

- Trenches should be installed at the end of the development construction;

- Smearing of the native material at the interface with the trench must be avoided and/or corrected by raking/roto-tilling; and

- Compaction of the trench during construction must be minimized.

Winter Operation

In general, infiltration facilities are unsuitable for water quality treatment during the winter/ spring period. They are subject to reductions in capacity due to freezing or saturation of the soil. If road runoff is received, there is an increased likelihood of clogging due to high sediment loads and an increased risk of groundwater contamination from road salt.

If infiltration practices are used as an all-season water quality treatment facility, then doubling the design storage volume for surface infiltration devices to account for reduced infiltration rates is recommended. Redundant pre-treatment (more than one pre-treatment device in series) is recommended for all infiltration facilities receiving road runoff. A pre-treatment volume of about 15 mm/impervious hectare is recommended.

Technical Effectiveness

Centralized infiltration trenches have a poor historical record of success (Lindsey et al., 1992; Metropolitan Washington Council of Governments, 1992). This lack of success is attributable to many factors:

- poor site selection (industrial/commercial land use, high water table depth, poor soil type);
- poor design (lack of pre-treatment, clogging by native material);
- poor construction techniques (smearing, over-compaction, trench operation during construction period); and
- large drainage area (high sediment loadings, groundwater mounding).

There are many reasons why an infiltration trench can fail. One of the main problems with centralized infiltration trenches is that water from a large area is expected to infiltrate into a relatively small area. This does not reflect the natural hydrologic cycle and generally leads to problems (groundwater mounding, clogging, compaction).

Water quality enhancement can be achieved using infiltration trenches. However, care must be taken to avoid degradation of groundwater quality. Trenches are ineffective quantity control facilities unless substantial storage is provided and the soil conditions are optimum.

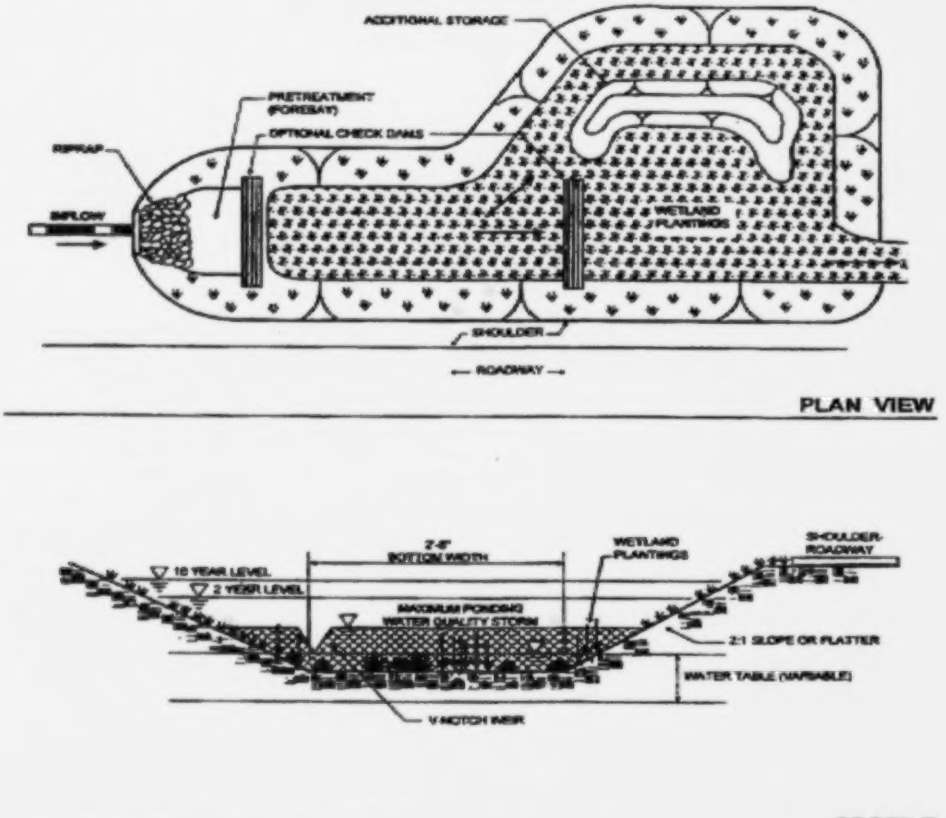
4.5.9 Grassed Swales

Grassed swales have historically been associated with rural drainage and have been constructed primarily for stormwater conveyance. Stormwater management objectives have changed and grassed swales are now being promoted to filter and detain stormwater runoff. Swale drainage can be a useful technique in areas of low grade, as long as the distance that the flow is to be conveyed is not too long.

The majority of swale systems in Ontario have been designed as "dry" swales. The guidance provided below is for such systems. An alternate design, the "wet" swale, can also be useful in areas where there is sufficient space, especially where soils are not highly permeable, or where there are low lying areas with a high water table.

Wet swales combine elements of dry swale systems and wetland systems. Wet swales are typically wider than dry swales (e.g., 4 m - 6 m) and the check dams are used to create shallow impoundments in which wetland vegetation is planted or allowed to colonize. Because of their width, wet swales are not generally implemented along the front of residential properties, but rather are included where overland flow routes use linear open space areas. Combined systems of dry and wet ponds may be used. Wet swales have been implemented in several highway projects, but monitoring results are limited. A schematic of a wet swale is provided in Figure 4.9.

Figure 4.9: Schematic of a Wet Swale


ADDITIONAL STORAGE
PRETREATMENT (FOREBAY)
OPTIONAL CHECK DAMS
RIPRAP
INFLOW
WETLAND PLANTINGS
SHOULDER
ROADWAY
PLAN VIEW
2'-8'
BOTTOM WIDTH
WETLAND PLANTINGS
SHOULDER-ROADWAY
10 YEAR LEVEL
2 YEAR LEVEL
MAXIMUM PONDING WATER QUALITY STORM
2:1 SLOPE OR FLATTER
WATER TABLE (VARIABLE)
V NOTCH WEIR
PROFILE

Wet swales are ideal for treating highway runoff in low lying or flat terrain areas.

Source: Maryland Stormwater Manual, Volume 1, 1998.

Design Guidance

Swale Cross-section

Grassed swales can be effective SWMPs for pollutant removal if designed properly. The water quality benefits associated with grassed swales depend on the contact area between the water and the swale and the swale slope. Deep narrow swales are less effective for pollutant removal compared to shallow wide swales. Given typical urban swale dimensions (0.75 m bottom width, 2.5:1 side slopes and 0.5 m depth), the contributing drainage area is generally limited to ≤ 2 ha (to maintain flow ≤ 0.15 m^3/s and velocity ≤ 0.5 m/s). Table 4.5 indicates drainage area restrictions for various degrees of imperviousness, based on the assumptions given regarding channel cross-section, slope and cover. The swales evaluated in Table 4.5 are indicative of swales servicing an urban subdivision and not a transportation corridor.

Table 4.5: Grassed Swale Drainage Area Guidelines*

% Imperviousness	Maximum Drainage Area (ha)
35	2.0
75	1.5
90	1.0

**Based on the following assumptions: trapezoidal channel, grassed lined (n = 0.035), slope of drainage area = 2%, 2.5:1 side slopes, 0.75 m bottom width, 0.5% channel slope, max. allowable Q = 0.15 m^3/s, max. allowable V = 0.5 m/s.*

Grassed swales are most effective for stormwater treatment when depth of flow is minimized, bottom width is maximized (≥ 0.75 m) and channel slope is minimized (e.g., ≤ 1%). Grassed swales with a slope up to 4% can be used for water quality purposes, but effectiveness diminishes as velocity increases. Grass should be allowed to grow higher than 75 mm to enhance the filtration of suspended solids.

Flow Velocity

As a general guideline, grassed swales designed for water quality enhancement should be designed to convey the peak flow from a 4 hour 25 mm Chicago storm with a velocity ≤ 0.5 m/s. This guideline results in a requirement for wide, flat swales for larger drainage areas.

All grass swales must be evaluated under major system and minor system events to ensure that the swale can convey these storms effectively.

Ditch and Culvert Servicing

Ditch and culvert servicing is viable for lots which will accommodate swale lengths ≥ the culvert length underneath the driveway (not just the driveway pavement width). The swale length should also be ≥ 5 m for aesthetic and maintenance purposes. This is generally achievable for small lots (9 m) with single driveways or larger lots (15 m) with double driveways.

Winter Operation

Swale systems which receive road runoff may have their infiltration capacity diminished over time, as salt effects on soil structure and clogging occur. Swale systems need to be maintained periodically (removal of accumulated sand and addition of mulch to the soil structure) in order to maintain their ability to infiltrate.

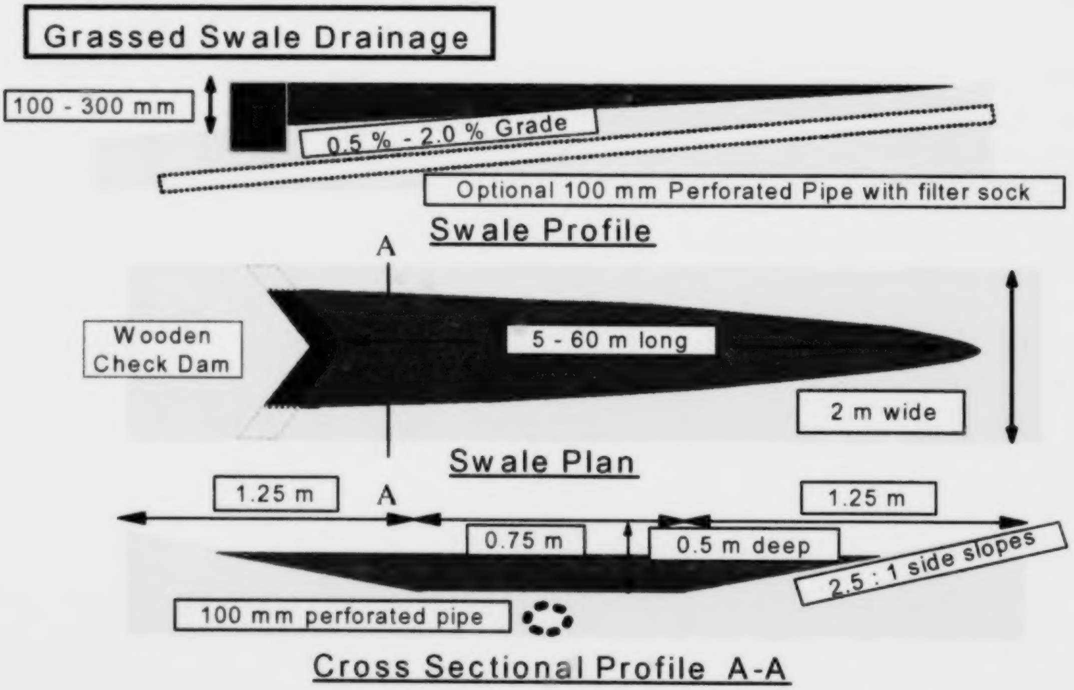
Relatively few design modifications are warranted for swales in cold climates, primarily due to their inherent simplicity. The following design modifications will tend to enhance their performance:

- Culverts should have a minimum diameter of 450 mm and a slope of 1% or greater; and
- For swale systems with an underdrain system, the underdrain should have a minimum diameter of 200 mm and should be bedded in gravel.

Performance Enhancements

In order to promote infiltration of stormwater and the settling of pollutants, permanent check dams can be constructed at intervals along the swale system. These enhancements are best utilized on large swales where the cumulative flow depth and rate is not conducive to water quality enhancement ($V \geq 0.5$ m/s or $Q \geq 0.15$ m^3/s during the 25 mm 4 hour storm). The distance between check dams can be calculated based on the depth of water at the check dam and the swale channel slope. For example, if a swale has a 1% slope and a check dam height of 0.3 m, the distance between check dams should be 30 metres (or less). Figure 4.10 illustrates an enhanced grassed swale design.

Figure 4.10: Enhanced Grass Swale


Grassed Swale Drainage
100 - 300 mm
0.5 % - 2.0 % Grade
Optional 100 mm Perforated Pipe with filter sock
Swale Profile
A
Wooden Check Dam
5 - 60 m long
2 m wide
Swale Plan
1.25 m
A
1.25 m
0.75 m
0.5 m deep
2.5 : 1 side slopes
100 mm perforated pipe
Cross Sectional Profile A-A

The dam should be constructed out of durable material (wood) which blends into the surrounding landscape. A rock check dam can be used if the swale is located in a remote area which is not subject to vandalism. The dam should be configured in a V shape to help minimize scour and erosion of the downstream swale banks (V points upstream). The dam should be securely embedded in the swale banks and some rip-rap should be placed downstream of the dam to prevent scour and erosion. The velocity of the design conveyance storm should be kept to approximately 1 m/s whereby smaller stone sizes can be utilized (75 mm diameter).

In areas where the swales are separated by driveway culverts, the culverts can be raised such that the driveway embankment (up to the invert of the driveway culvert) acts as the check dam. This design is more aesthetically appealing and negates the need for rip-rap erosion protection. The driveway culvert should be underdrained, however, to ensure that a permanent pool of water is not created in the swale.

A low flow opening can be created in the check dam to ensure a drawdown time ≤ 24 hours. However, recognizing the potential for clogging of the low flow opening, it is recommended that swales with check dams be underdrained in soils with poor infiltration potential (e.g., clays).

Standard 100 mm perforated pipe (or larger) should be used in combination with a filter sock in any type of underdrain system. Stone storage can be provided around perforated pipes that are installed under swales as a secondary storage medium to promote exfiltration. The appropriate depth of soil cover for the stone storage should be based on the surrounding soil conditions and the potential for frost heave. Figure 4.4 indicates the recommended soil cover based on the native soil type and trench depth.

All grass swales must be evaluated under major system and minor system events neglecting the storage/conveyance below the overflow of any check dam to ensure that the swale can convey these storms effectively.

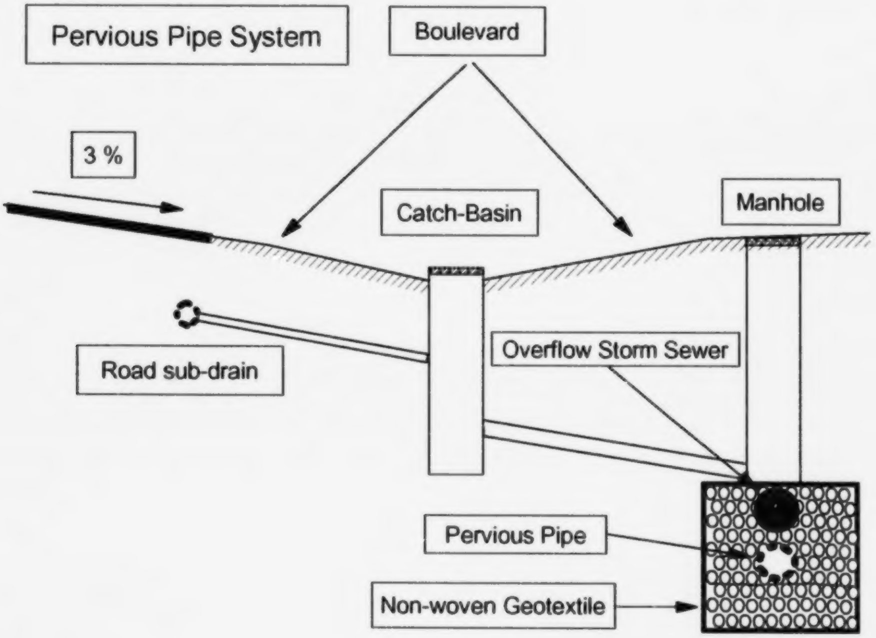
Technical Effectiveness

The effectiveness of swale systems is highly dependent on their design and maintenance. It is therefore recommended that they be used as part of a multi-component approach (i.e., one measure in a series of stormwater quality measures). They may be used for pre-treatment or polishing.

4.5.10 Pervious Pipe Systems

A few municipalities in Ontario (e.g., City of Nepean, City of Etobicoke) have implemented pervious pipe systems (Figure 4.11). These systems have experienced some problems in the past and are still experimental in nature.

Figure 4.11: Pervious Pipe System



Pervious Pipe System
Boulevard
3 %
Catch-Basin
Manhole
Road sub-drain
Overflow Storm Sewer
Pervious Pipe
Non-woven Geotextile

Pervious pipe systems are perforated along their length allowing exfiltration of water through the pipe wall as it is conveyed downstream. The pipe itself is similar to that used for tile drainage on agricultural lands and is available with either a smooth-walled or corrugated interior.

Design Guidance

Soils

Pervious pipes can be used where soils have a percolation rate ≥ 15 mm/h.

Water Table Depth

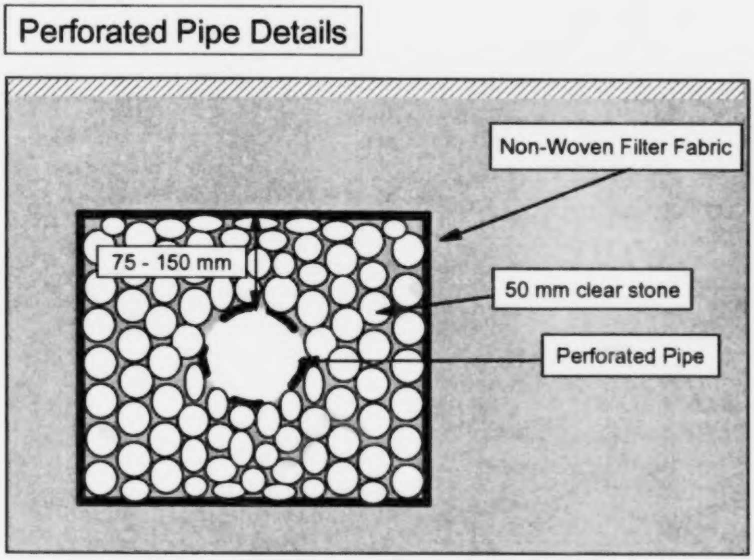
If a pervious pipe system is implemented in an area where the seasonally high water table is higher than the obvert of the pipe, the pipe will drain the groundwater table. In this scenario, depending on the native soil characteristics and whether the trench or pipe is wrapped in geotextile fabric, soil can be transported into the pipe system undermining the pipe foundation and leading to structural failure. Pervious pipe systems should not be implemented in areas where the seasonal high groundwater level is within 1 metre of the bottom of the storm sewer backfill to ensure this does not happen.

Depth to Bedrock
The depth to bedrock should be greater than or equal to 1 metre below the bottom of the perforated pipe storage media to ensure adequate drainage/hydraulic potential.

Storage Volume
A minimum storage volume equal to the runoff from a 4 hour 5 mm storm over the contributing drainage area should be accommodated in the pervious pipe bedding/storage medium without overflowing. The maximum target storage volume should be equal to the runoff from a 4 hour 15 mm storm over the contributing drainage area since 80% of all daily rainfall depths are less than this amount.

Storage Configuration
The exfiltration storage bedding layer should be 75 mm - 150 mm deep above the pervious pipe. A shallow bedding above the pipe is used since the storage above the pipe obvert is not utilized. The depth of bedding below the pipe obvert is dependent on the storm to be exfiltrated and the native soil material. Maximum depths which permit the bedding to drain in 24 hours can be calculated using Equation 4.2. Details of a pervious pipe end section are shown in Figure 4.12.

Figure 4.12: Pervious Pipe Details



The width of storage can be determined based on the rate of exfiltration from the pipe, the proposed length of pervious pipe, and the desired volume of runoff for infiltration. Section 4.8 provides methods for estimating the rate of exfiltration based on the number and size of pipe perforations.

Pipe Slope

Pervious pipe systems should be implemented with reasonably flat slopes (0.5%) to promote exfiltration.

Pervious Pipe Bedding/Storage Media

Granular A material, or preferably clear stone (50 mm), should be used for the pipe bedding. Granular B is generally discouraged for use as pervious pipe bedding because it contains too many fines which may infiltrate the pipe system.

Pervious pipe systems should be constructed with anti-seepage collars to ensure that exfiltrated water does not travel along the pervious pipe bedding to the outlet. The spacing of anti-seepage collars should be based on the permeability of the native soil material and the pipe slope.

Pervious Pipe

Smooth-walled (interior) pervious pipe is recommended for stormwater exfiltration since corrugated pipe has a higher potential for clogging. Furthermore, the maintenance of corrugated pipe via traditional sewer flushing is relatively ineffective since material becomes trapped in the corrugations. A minimum diameter of 200 mm should be used for the pervious pipe to facilitate maintenance.

Geotextiles

Although a filter sock can be used to prevent fines from entering the pipe system from the native material, the sock may prevent fines in stormwater from exfiltrating to the native material. As such, the use of a filter sock may cause clogging at the pipe/sock interface and decrease the longevity of the pervious pipe system.

Non-woven filter fabric installed at the interface between the pipe bedding (exfiltration storage) and the native soil can prevent native material from clogging the voids in the exfiltration storage media.

Pre-treatment

Pervious pipe systems are intended to convey road drainage which has high levels of suspended sediment. Pre-treatment of road drainage is necessary before it reaches the pervious pipe system to enhance the longevity of the system and reduce the potential for groundwater contamination.

Pre-treatment of pervious pipe systems can best be achieved by the incorporation of grassed boulevards as pre-treatment areas (Figure 4.11). Stormwater is conveyed from the road to a low boulevard. The boulevard is graded towards catchbasins which are connected to the pervious pipe system. The catchbasins are raised such that water must reach a certain depth in the boulevard before it can overflow into the pervious pipe system. This will provide a sense of an urban cross-section while maintaining the benefits of traditional grass surfaced conveyance systems.

Technical Effectiveness

Pervious pipe systems have been implemented in a number of municipalities. In areas where they have been implemented and monitored, numerous systems clogged after several years. The Regional Municipality of Ottawa-Carleton, however, has reported success with pervious pipe systems. Pervious pipe systems can be an effective alternative in retrofit situations (e.g., replacement of existing storm sewers), especially in areas where the catchment has stabilized.

The primary reason for system failure is clogging which can be attributed to several factors:

- poor design (storage media, lack of filter cloth, lack of pre-treatment);
- poor construction practices;
- inadequate stabilization of development before implementation of pervious pipe (construction timing); and
- poor site physical conditions (soils, water table).

One of the problems of implementing pervious pipe systems is construction timing. Ideally, for new development the pervious pipe system would be constructed after the houses have been built and the sod has been laid. However, the road sub-grade needs to be drained and requires the pipe system to be constructed with the road network. The pervious pipe system functions as the storm sewer and therefore must be constructed in its entirety. Although the catchbasins can be blocked to try and prevent sediment laden water from clogging the pervious pipe system, there is a great potential for clogging and compaction of the system during the construction phase of development.

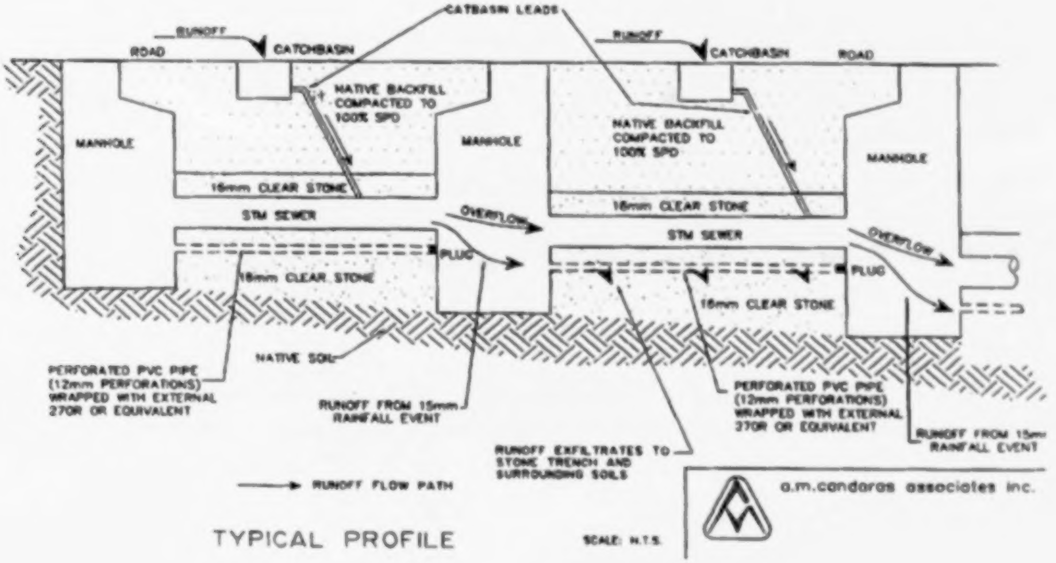
Pre-treatment of road drainage before it reaches the pervious pipe system will enhance the longevity of the system and reduce the potential for groundwater contamination.

Etobicoke Exfiltration System

The former City of Etobicoke (now City of Toronto) implemented a double pipe system (regular storm sewer over a perforated pipe) in a retrofit situation on a local road which is not subject to heavy salting or sanding. This system, while relatively expensive if applied in a "new development" situation, provides a means for implementing water quality controls on a retrofit basis in areas of existing development which are undergoing storm sewer rehabilitation or upgrading.

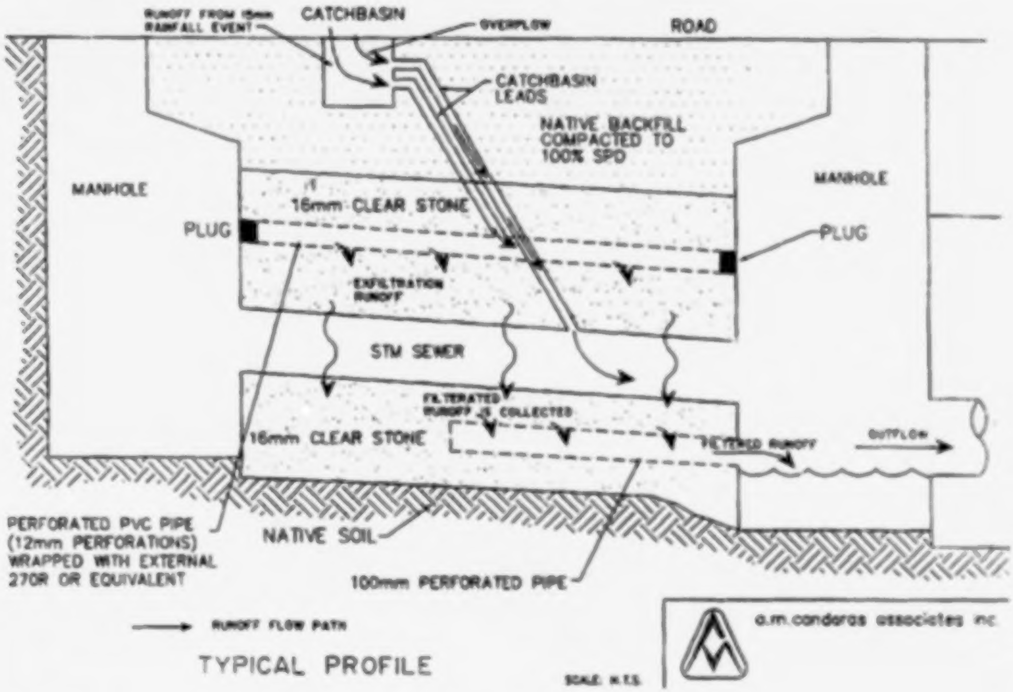
In the Etobicoke system, road runoff is captured in catchbasins and fed into the conventional storm sewer pipe. At the next downstream manhole, flow drops down into a perforated pipe which is plugged at its downstream end. Runoff either exfiltrates, or if the capacity is exceeded, backs up into the conventional storm sewer which conveys it to the next manhole, and eventually to its outlet. The two pipe system provides a contingency conveyance system if the perforated pipe becomes clogged. A double pipe system also allows the perforated pipe to be plugged during the construction phase until the site has stabilized, thereby preventing it from becoming clogged prematurely. The exfiltration system is illustrated in Figure 4.13.

Figure 4.13: Exfiltration System


CATBASIN LEADS
RUNOFF
ROAD
CATCHBASIN
RUNOFF
CATCHBASIN
ROAD
NATIVE BACKFILL COMPACTED TO 100% SPD
MANHOLE
MANHOLE
NATIVE BACKFILL COMPACTED TO 100% SPD
MANHOLE
16mm CLEAR STONE
16mm CLEAR STONE
STM SEWER
OVERFLOW
STM SEWER
OVERFLOW
PLUG
PLUG
16mm CLEAR STONE
16mm CLEAR STONE
NATIVE SOIL
PERFORATED PVC PIPE (12mm PERFORATIONS) WRAPPED WITH EXTERNAL 270R OR EQUIVALENT
RUNOFF FROM 15mm RAINFALL EVENT
PERFORATED PVC PIPE (12mm PERFORATIONS) WRAPPED WITH EXTERNAL 270R OR EQUIVALENT
RUNOFF FROM 15mm RAINFALL EVENT
RUNOFF EXFILTRATES TO STONE TRENCH AND SURROUNDING SOILS
RUNOFF FLOW PATH
a.m.candaras associates inc.
TYPICAL PROFILE
SCALE: N.T.S.

The exfiltration system is best implemented in areas with pervious soils. A variation on the system uses filtration rather than exfiltration and is applicable to areas with tighter soils. In this variation, flow from the catchbasin is discharged to a length of perforated pipe within a gravel-filled trench (in which the conventional storm sewer is also bedded). The runoff filters down through the trench and is collected by a second perforated pipe at the bottom of the trench. The second pipe conveys flow to the next downstream manhole and into the conventional sewer system. If the trench volume or catchbasin capacity is exceeded, a second, higher level outlet in the catchbasin allows flow to be conveyed to the conventional storm sewer. This configuration is illustrated in Figure 4.14.

Figure 4.14: Filtration System


RUNOFF FROM 15mm RAINFALL EVENT
CATCHBASIN
OVERFLOW
ROAD
CATCHBASIN LEADS
NATIVE BACKFILL COMPACTED TO 100% SPD
MANHOLE
MANHOLE
16mm CLEAR STONE
PLUG
PLUG
EXFILTRATION RUNOFF
STM SEWER
FILTERATED RUNOFF IS COLLECTED
16mm CLEAR STONE
FILTERED RUNOFF
OUTFLOW
PERFORATED PVC PIPE (12mm PERFORATIONS) WRAPPED WITH EXTERNAL 270R OR EQUIVALENT
NATIVE SOIL
100mm PERFORATED PIPE
RUNOFF FLOW PATH
TYPICAL PROFILE
SCALE: N.T.S.
a.m.candaras associates inc

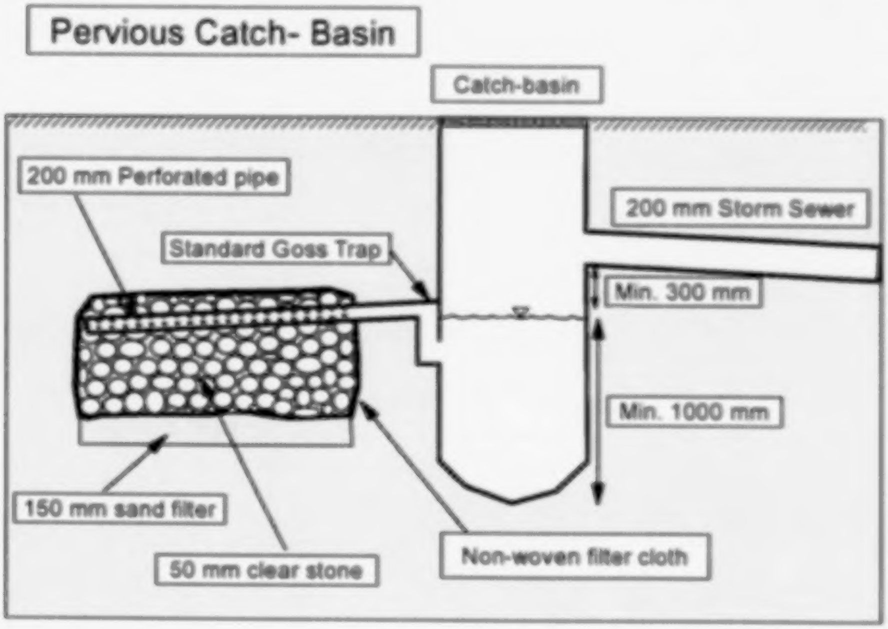
A monitoring program for the Etobicoke systems has been completed (A.M. Candaras Associates Inc., 1997). The study was sponsored by the City of Etobicoke, MOE and the Great Lakes Clean-up Fund (now referred to as the Government of Canada's Great Lakes Sustainability Fund). Two exfiltration systems (serving areas of about 13 and 30 hectares) and a filtration system (serving 2.4 hectares) were monitored. The systems each work very well and may in fact be overdesigned. The design basis for the systems was the runoff from a 15 mm Atmospheric Environment Service (AES) storm and exfiltration to saturated media. There have been no reported overflows of the perforated pipe system and it has been hypothesized that much higher rates of exfiltration are occurring because the media is not saturated. Since there were no outflows from the system, contaminant discharge was eliminated over the period of monitoring.

4.5.11 Pervious Catchbasins

Pervious catchbasins are simply normal catchbasins with a larger sump which are physically connected to an exfiltration storage medium. In some designs, the storage medium is connected to the catchbasin located directly above via a hole or series of holes in the catchbasin floor. Although this design is convenient and conserves land, it is more susceptible to clogging and compaction as a result of the lack of pre-treatment and the weight of the water in the catchbasin. There are manufacturers which offer catchbasin filters for pre-treatment in this type of design. These filters are expensive, however, and need frequent replacement.

A second design (Figure 4.15) uses the catchbasin sump for pre-treatment of runoff and discharges low flows through the wall of the catchbasin to the adjacent exfiltration storage medium.

Figure 4.15: Pervious Catchbasin


Pervious Catch- Basin
Catch-basin
200 mm Perforated pipe
200 mm Storm Sewer
Standard Goss Trap
Min. 300 mm
Min. 1000 mm
150 mm sand filter
50 mm clear stone
Non-woven filter cloth

Design Guidance

Soils

Pervious catchbasins may be used where soils have a percolation rate > 15 mm/h.

Water Table Depth

Pervious catchbasins should not be implemented in areas where the seasonal high groundwater level is within 1 metre of the bottom of the infiltration trench.

Depth to Bedrock

The depth to bedrock should be greater than or equal to 1 metre below the bottom of the infiltration trench to ensure adequate drainage/hydraulic potential.

Storage Volume

A minimum storage volume equal to the runoff from a 4 hour 5 mm storm over the contributing drainage area should be accommodated in the pervious pipe bedding/storage medium without overflowing. The maximum target storage volume should be equal to the runoff from a 4 hour 15 mm storm over the contributing drainage area since 80% of all daily rainfall depths are less than this amount.

Storage Configuration

The exfiltration storage depth is dependent on the native soil type/characteristics. Maximum depths can be calculated based on the native soil percolation rate and Equation 4.2. The length and width will depend on the area of land available for the trench (up to the maximum storage volume equal to the runoff from a 15 mm storm over the contributing area).

Storage Media

Clear stone (50 mm) should be used as the exfiltration storage medium (porosity = 0.4).

Geotextile

Non-woven filter fabric should be installed at the interface between the exfiltration storage and the native soil to prevent native material from clogging the voids in the exfiltration storage medium.

Pre-treatment

Pervious catchbasins are intended to infiltrate road drainage which has high levels of suspended sediment. Exfiltration of stormwater without pre-treatment will result in poor longevity of the exfiltration system. Large catchbasins with deep sumps will help pre-treat the runoff before it is conveyed to the infiltration trench. However, the amount of pre-treatment will be small even for large manholes, and other pre-treatment measures should be incorporated, if possible, before the stormwater enters the sewer system. Pre-treatment is best achieved by the incorporation of grassed boulevards as discussed in the previous section on pervious pipes.

Technical Effectiveness

Pervious catchbasins have been used in both the Cambridge and the Ottawa areas. As with the pervious pipe systems, varying results have been reported. The Regional Municipality of Ottawa-Carleton has reported success with pervious catchbasins. Where difficulties have been observed, it has usually been due to:

- poor design (storage media, filter cloth, lack of pre-treatment);
- poor construction practices;
- inadequate stabilization of development before construction (construction timing); and
- poor site physical conditions (soils, water table).

One of the benefits of pervious catchbasins which are located off-line is that they can be plugged until construction has finished and the development has been stabilized. This helps to prolong the life of the exfiltration storage.

Pre-treatment of road drainage before it reaches the pervious catchbasins will enhance the longevity of the system and reduce the potential for groundwater contamination. Frequent catchbasin cleaning is required to ensure the longevity of this SWMP. Eventually, the exfiltration storage will become clogged and need to be replaced.

4.5.12 Vegetated Filter Strips

Vegetated filter strips are engineered stormwater conveyance systems which treat small drainage areas. Generally, a vegetated filter strip consists of a level spreader and planted vegetation. The level spreader ensures uniform flow over the vegetation which filters out pollutants, and promotes infiltration of the stormwater.

There are two types of vegetated filter strips: grass filter strips, and forested filter strips. There is a need for further research comparing the efficiency of these two systems for water quality enhancement, since the research to date has focussed on their individual assessment.

Vegetated filter strips are best utilized adjacent to a buffer strip, watercourse or drainage swale since the discharge will be in the form of sheet flow, making it difficult to convey the stormwater downstream in a normal conveyance system (swale or pipe).

Design Guidance

Drainage Area

Vegetated filter strips are feasible for small drainage areas (< 2 ha).

Slope and Width

Vegetated filter strips should be located in flat areas (< 10%) to promote sheet flow and maximize the filtration potential. The ideal slope in a vegetated filter strip is < 5% (1% - 5%).

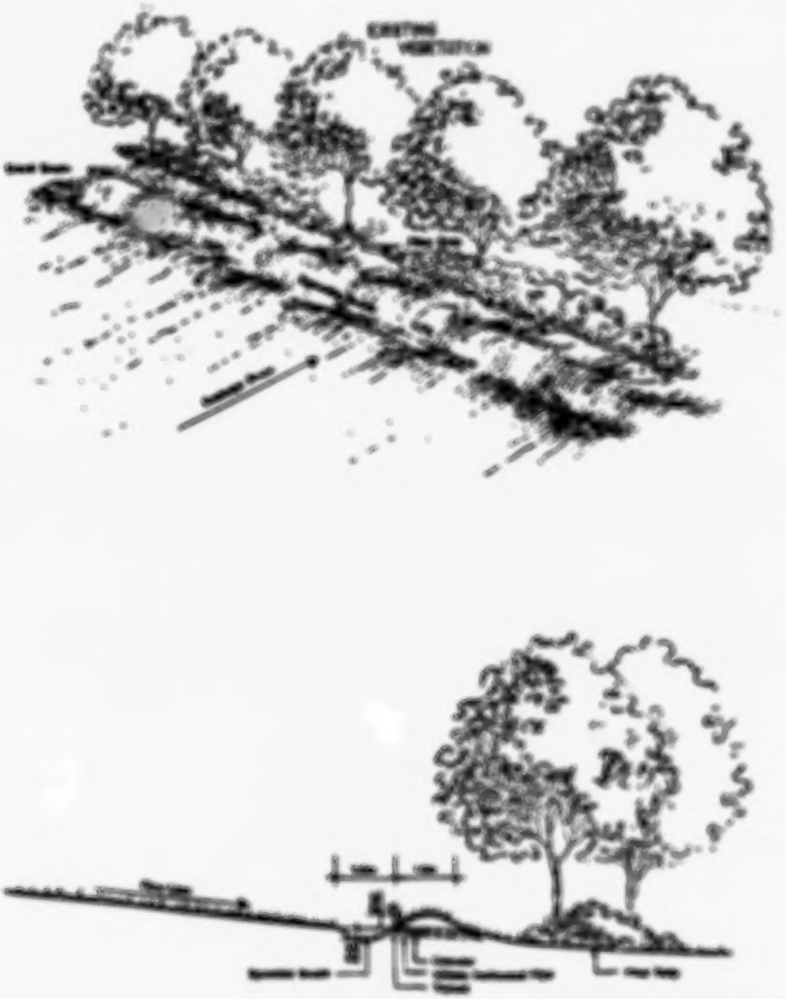
The vegetated filter strip should be 10 m - 20 m wide in the direction of flow to provide sufficient stormwater quality enhancement (Osborne et al., 1993; Metropolitan Washington Council of Governments, 1992; Minnesota Pollution Control Agency, 1989). The slope of the vegetated filter strip should dictate the actual width. Shorter vegetated filter strip widths (10 m - 15 m) are appropriate for flat slopes, whereas longer vegetated filter strips (15 m - 20 m) are required in areas with a higher slope (5% - 10%).

Level Spreader

The level spreader consists of a raised weir constructed perpendicular to the direction of flow. Water is conveyed over the spreader as sheet flow to maximize the contact area with the vegetation. Although the spreader can be engineered using concrete, more natural spreader designs/materials are recommended to maintain a natural appearance.

Figure 4.16 illustrates a typical level spreader design. A small berm is used as the level spreader. It creates a damming effect, preventing stormwater from entering the vegetation until the water level exceeds the height of the spreader. A perforated pipe (100 mm diameter) is installed in the spreader berm to ensure that any water which is trapped behind the berm after a storm can be drained. The perforated pipe should be wrapped in a filter sock to ensure that native material does not infiltrate the pipe.

Figure 4.16: Typical Filter Strip



The length of the level spreader should be chosen based on site specifics (topography, outlet location, drainage area configuration). It should be recognized, however, that a shorter level spreader necessitates the trade-off of greater upstream storage to maintain the desired flow depth over the vegetation. It is recommended that the level spreader length, and hence vegetated filter strip length, be as large as possible.

Flow Depth

The level spreader and vegetated filter strip should be designed such that the peak flow from a 4 hour Chicago 10 mm storm results in a flow depth of 50 - 100 mm through the vegetation. The flow depth over the level spreader can be calculated using a standard broad crested weir equation (Equation 4.4).

$$Q = \alpha L H^{1.5}$$

Equation 4.4: Weir Flow

where Q = discharge
α = coefficient
L = length of crest of weir
H = head

Storage

Storage will be required behind the level spreader depending on the level of control desired, and the length of the level spreader itself. The amount of storage required should be based on the excess runoff from a 4 hour Chicago distribution of a 10 mm storm, accounting for the flow over the weir. The 10 mm storm was chosen recognizing that 70% of all daily precipitation depths are less than or equal to this amount.

Vegetation

Species such as red fescue, tall fescue and redtop can be introduced in addition to the natural surrounding vegetation to filter out stormwater pollutants. Species native to the area should be used, where commercially available, in the planting strategy.

Technical Effectiveness

Vegetated filter strips have limited effectiveness for water quality control due to the difficulty of maintaining sheet flow (i.e., preventing channelization) through the vegetation. They are best implemented as one in a series of SWMPs in a stormwater management plan.

4.5.13 Stream and Valley Corridor Buffer Strips

Buffer strips are simply natural areas between development and the receiving waters. There are two broad resource management objectives associated with buffer strips:

- The protection of the stream and valley corridor system to ensure their continued ecological form and functions; and

- The protection of vegetated riparian buffer areas within the valley system to minimize the impact of development on the stream itself (filter pollutants, provide shade and bank stability, reduce the velocity of overland flow).

Although both types of buffers provide only limited benefits in terms of stormwater management, they are an integral part of overall environmental management for sustainable development. The protection of stream and valley corridors provides significant benefits in terms of sustaining wildlife migration corridors, terrestrial and aquatic species food sources, terrestrial habitat, and linkages between natural areas.

Given the larger scale natural system benefits provided by stream and valley corridors, the required width of this type of buffer is best defined at the subwatershed plan level. Individual conservation authorities and municipalities have developed their own guidelines for buffer areas. The designer should confirm local requirements with the applicable authority.

4.5.14 Roof Top Gardens

Roof top gardens are regarded as a relatively recent innovation in the field of stormwater management, including quality improvement. Roof top gardens may be as simple as a sodded roof which retains water in the soil medium and provides filtration, to something as elaborate as a fully landscaped area with trees, shrubs, gardens, fountains, seating areas and other outdoor amenities. The range of plants suitable for use in roof top landscapes is limited by the extremes of microclimate including high wind, low winter temperature (i.e., no moderating effect from heat stored in the ground) and drought. As a result, alpine or sub-alpine species are well suited to roof top applications. In more elaborate schemes, infrastructure such as irrigation systems, increased insulation and venting from interior heat sources can be employed to overcome the constraints of microclimatic conditions. Roof top gardens have been used extensively and successfully in Europe and their performance is well documented. On this basis, the roof top garden should be considered as a viable stormwater management option which may be used to reduce the scale of associated end-of-pipe treatment facilities within the context of an overall development.

4.6 End-of-Pipe Stormwater Management Facilities

4.6.1 SWMP Vegetation

Vegetation is an integral part of many SWMPs. Much of the guidance applies to several of the end-of-pipe practices which are described in subsequent sections. The considerations and approaches common to various SWMPs are provided in this section. Where appropriate, details specific to individual SWMP types are provided with other design guidance.

4.6.1.1 Effective Use of Vegetation

Vegetation should be considered as an important functional component in the design of SWMPs including ponds, wetlands, vegetated filter strips and bioretention filters. Vegetation filters stormwater and takes up nutrients, and in wet facilities, it promotes settling by reducing flow velocities and preventing re-suspension. In addition to enhancing water quality, vegetation effectively achieves the following:

The Stabilization of Banks, Shoreline and Slopes
The rooting systems of many species of trees, shrubs and herbaceous plants effectively bind soils to establish a layer that is resistant to erosion. Planting schemes that combine plant species selected for their unique and complementary rooting characteristics are typically most effective in providing long-term stability.

Mitigation of Effects on Temperature and Dissolved Oxygen
The strategic location of deciduous and coniferous trees along the edges of a pond, channel or wetland can assist in mitigating undesirable increases in water temperature. In addition, vegetation can contribute to the maintenance of dissolved oxygen levels by inhibiting the growth of algae.

Deterrence of Geese
The establishment of dense woody vegetation around the perimeter of a pond or wetland is the most effective means of deterring undesirable species of waterfowl from colonizing and contaminating facilities which have a permanent pool. Minimizing the amount of manicured/mown land will limit the preferred habitat for geese.

Provision of Barriers to Mitigate Public Access
Thickets of thorn bearing shrubs and trees, and twining vines can be combined to create an impenetrable barrier to deter the public from accessing pond areas, steep slopes and other areas which are deemed potentially hazardous.

Enhancement of Linkages
The establishment of diverse communities of plants in conjunction with a SWMP can contribute to the establishment of linkages between natural wooded areas, providing terrestrial habitat benefits at a larger scale.

Provision of Aesthetic Benefits
Vegetation can be utilized to create visual buffers, enhance views and contribute to the establishment of a unique character for a development. Vegetation is one of the most effective tools to blend a SWMP into its surroundings from a visual perspective.

Additional Benefits
Vegetation can also be utilized in the design of SWMPs to achieve the following:

- Intercept rainfall;
- Filter out coarse sediments;
- Trap and accumulate floatables;
- Reinforce and maintain the integrity of spreaders, weirs and retaining walls;

- Intercept airborne pollutants;
- Impede colonization by undesirable invasive species; and
- Conceal fencing and structures.

Careful selection of plant material is the critical factor to ensure that functional objectives are achieved. The determination of plant species best suited to a specific application should be made based upon the following considerations:

- Regional climatic characteristics/Hardiness Zone;
- Microclimate conditions;
- Frequency and duration of flooding;
- Soil conditions;
- Environmental features;
- Plant community composition;
- Proximity to roads and other potential sources of airborne pollutants and salt spray;
- Maintenance requirements; and
- Availability of nursery stock/opportunities for transplantation.

The following principles should be applied to guide the selection of plant material and the generation of planting strategies for SWMPs.

Plan for Succession

Vegetation communities are dynamic, evolving over time to adapt to the changing environment. Planting design must recognise this evolutionary process to ensure that objectives are achieved over the long term.

Limited monitoring results show that species may shift from those planted to those which are more locally successful, particularly within pools and frequently inundated areas. Planting strategies are important in ensuring effective SWMP operation; however, they do not need to be overly complex since natural succession plays an important role in the ultimate make-up of the vegetation community.

Design to Enhance Ecological Function

Although it may not be an objective to create terrestrial and aquatic habitat in the design of a SWMP, through objective planting design, numerous other ecological objectives can be achieved, including the establishment of linkages, increase in canopy cover, provision of food and shelter and modification of microclimate which will enhance the integrity of the ecosystem at a regional scale.

Have Regard for Context

An understanding of the ecological, physical and social context of a site will help to direct the selection of appropriate plant species and the assembly and configuration of plant communities which are appropriate to the site and provide the maximum benefit.

Utilize Species which are Indigenous to the Bioregion
Since SWMPs are linked to a network of habitats which are connected by the watercourse downstream of the facility, it is important that plant material within the SWMP site be not only native but indigenous to the bioregion. The introduction of non-native, invasive species can threaten plant communities throughout the watershed and must be avoided.

Maximize Diversity
The use of a wide range of native, indigenous trees, shrubs, wildflowers, grasses, sedges and aquatic plants will not only enhance biodiversity on a watershed-wide scale, but also contribute to the system's resiliency and ability to maintain itself. Plants such as cattail and common reed are aggressive and may thwart diversity goals.

Recognize Human Factors
Planting design should be developed with a recognition of the requirements of the adjacent residents, users of the site and the community-at-large related to recreational requirements, interpretive opportunities, aesthetics, public safety issues and other associated factors.

4.6.1.2 Developing a Planting Strategy

Appendix E provides a catalogue of species of trees, shrubs, vines and herbaceous materials which are appropriate for use in the design of ponds, wetlands and other SWMPs. Depth and frequency of inundation, particularly during the growing season, are the primary factors controlling species survival. Water quality may be a secondary consideration. The planting strategy may include up to five zones based on frequency of inundation:

- deep water areas;
- shallow water areas;
- extended detention or shoreline fringe areas;
- flood fringe areas; and
- upland areas.

Some species readily cross category boundaries and many species can survive in the near reaches of neighbouring zones. Category boundaries will differ regionally. Different genotypes of a species may have different ranges of tolerance depending upon genetic and physiological adaptations. The lists in Appendix E may be used as a guide but species selection should also be based on observations of natural systems in the area and sources of plant material. Also keep in mind that new plantings will be less tolerant of extremes than mature individuals.

In natural systems, grasses tend to occur in areas with shallow or limited seasonal flooding. As frequency of inundation and water depth increase, grasses give way to sedges, rushes and spikerushes. These in turn are replaced by broad-leaved emergents such as pickerel weed and arrowhead. Cattails may be found with these plants but also may extend into deeper water where large bulrushes occur. In deep water, submergents will grow provided water clarity is suitable. In the deepest regions, or where water is coloured or turbid, rooted floating plants such as water lilies may be found. Only a few shrubs and trees, such as green ash, buttonbush, black willow, and red osier dogwood, can withstand prolonged inundation.

Deep Water Areas

Aquatic species, including submergents and floating-leaved species, are appropriate for deep water areas (> 0.5 m). Rooted aquatic plants with floating leaves (e.g., pondweeds) and free floating plants (e.g., duckweed) will grow in the deepest areas. Some emergent species (e.g., cattail and bulrush) may tolerate water depths greater than 0.5 m and could be planted in the shallowest area of the deep water zone.

Shallow Water Areas

Submergent and emergent vegetation may be used in the shallow water (< 0.5 m) zone. Most emergents should be planted at a water depth < 0.3 m. Minimum side slopes are preferable to maximize the area available for plantings.

Shoreline Fringe Areas

Shoreline fringe areas are subject to frequent wetting as a result of storm events. This zone can be delineated as the land between the permanent pool, or the pond bottom in dry ponds, and the high water mark for the erosion/water quality control storage. This zone will be subject to higher soil moisture conditions as a result of water level fluctuations during relatively frequent storm events, and the influence of the permanent pool itself in wet facilities during dry weather conditions.

Many wetland sedges, rushes, wildflowers, ferns and shrubs may be planted in the shoreline fringe areas of wet ponds and wetlands. The growing conditions in dry ponds are harsher since there is no influence from a permanent pool. A grass seed mixture can be sown in the fall, although spring is preferable. The shrubs should be planted such that only their lower branches will be inundated during the design storm. At least two shrub species should be planted to improve survival success.

Flood Fringe Areas

If the wet pond is used to control peak flow rates during infrequent storm events (2 year to 100 year), a zone of infrequent inundation will be created. The influence of a permanent pool and frequent storm events are less pronounced for this area. The planting strategy in this zone may include a variety of grass, herb, shrub, and tree species. A commercially available grass and herb seed mixture suitable for slope stabilization is recommended. The grass and herb seed mixture should be sown in the fall or preferably the spring. At least three species of shrubs and three species of trees should be planted in this zone. There should be a gradual change in planting near the upland zone for aesthetic reasons.

In many designs, as a safety feature and alternative to fencing, thorny vegetation (such as hawthorn or raspberry) is planted in the flood fringe zone. These plants, together with the shallow water and shoreline fringe plantings, act as a barrier to casual entry. Care should be taken, however, that fast growing vines and brambles do not smother young trees.

Upland Areas
Upland areas represent the landscaped areas provided as aesthetic amenities around the pond. Upland planting should also be designed to restrict access to steep areas or inlet/outlet locations. At least 5 species should be planted in a random pattern to prevent the establishment of monoculture areas. A large number of young plant stocks, tree whips and seedlings should be planted rather than a small number of large shrubs and trees. Some mature plants should be used, however, to meet immediate wind screening, shading, aesthetic and safety objectives. The upland planting should provide a minimum of a 3 m buffer strip from the maximum design water level mark. The massing of trees and shrubs should be augmented by designated regeneration areas to achieve long-term growth. A naturalized landscape approach should be used which strives for a vegetation community with long-term sustainability and no maintenance requirements.

4.6.1.3 Planting Techniques

Loam soils normally have adequate nutrients, provide good water and gas circulation, and have an intermediate texture that supports new plants but allows root or rhizome penetration. Within the permanent pool, 0.3 m of topsoil should be provided to a water depth of 1 m. In the active storage and upland areas, 0.45 m - 1 m of topsoil is needed. It should be stabilized by seeding and engineered methods such as erosion control mats may also be required.

The choice of planting technique is strongly influenced by the plant species selected. It is possible to create suitable conditions for natural invasion and establishment if there is a nearby seed source or the substrate contains a seed bank. It may take a number of years to establish vegetation using this method and the ability to control the type and distribution of plants will be limited.

Grasses are more easily established by seeding than other plant types. Seed germination rates may be quite low for some species, such as some emergent plants that reproduce and spread largely by vegetative means. Various conditions are required for germination of different species. Success may depend upon the ability to establish optimal conditions for seed germination by managing water levels. Optimal conditions for many species will involve shallow flooding and subsequent dewatering or maintaining water levels just below surface. As plants grow, shallow flooding may be desirable to inhibit the growth of competing terrestrial species, but deep flooding may also stress wetland species and overtopping individual plants may result in mortality.

Above the permanent pool, ground cover may be established by hydroseeding or using a custom seed mix in a soil nutrient medium. Protection of the substrate and seed using a biodegradable blanket is recommended particularly in the zone of dynamic water level fluctuation.

Tubers or rhizomes may be planted after dormancy in the fall or in the early spring before the growing season starts. Tubers are forced into soft substrates deep enough to prevent them from floating out and rhizomes are inserted into slits or trenches angled slightly upward.

Whole plants have the advantage of an established root structure for early stability. Shoots at least 10 cm tall reduce mortality due to overtopping but should not exceed 25 cm to reduce the threat of windthrow before the roots have an opportunity to anchor the plant in the substrate. They should be planted in the spring. Bare root seedlings are more susceptible to transplanting shock than containerized plants that have their roots in a suitable growth medium. Refer to the Native Plant Resource Guide for Ontario (Society for Ecological Restoration, 2001) for suppliers of various plants and stock types (seed, seedlings, containers).

Plant spacings of 0.75 m to 1.5 m are commonly used. Decreasing the spacing reduces the time required to achieve complete coverage. The spacing between trees will be larger (5 m - 6 m). In shallow water zones, the planting rows should be perpendicular to the direction of the water flow to minimize the potential for channelization.

Ideally, vegetation should be allowed to overcome planting shock before being subjected to the stresses of flooding and contaminant loadings. Herbaceous species may take up to several years to become well established and woody species may take much longer. During the start-up period, the vegetation should be monitored frequently. If areas of vegetation appear to be unhealthy or dead, replanting may be required.

4.6.2 Wet Ponds

Wet ponds are the most common end-of-pipe stormwater management facility employed in Ontario. They are less land-intensive than wetland systems and are normally reliable in operation, especially during adverse conditions (e.g., winter/spring) This reliability can be attributed to several factors:

- performance does not depend on soil characteristics;
- the permanent pool minimizes re-suspension;
- the permanent pool minimizes blockage of the outlet;
- biological removal of pollutants occurs; and
- the permanent pool provides extended settling.

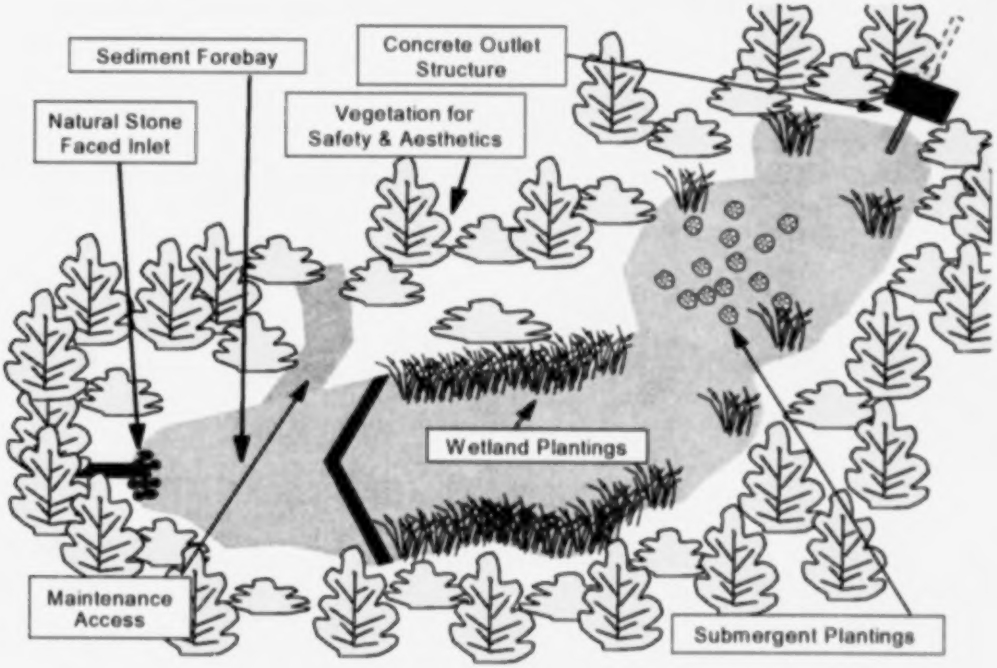
Wet ponds can be designed to efficiently provide for water quality, erosion and quantity control, reducing the need for multiple end-of-pipe facilities. Wet ponds can be designed with extensive landscaping and associated recreational amenities, contributing to the character of the community and enhancing its marketability.

Design Guidance

A good design for a wet pond involves attention to a variety of criteria. In current practice the emphasis tends to be on volumetric aspects of design. However, the operational success of a wet pond is often dependent upon other design elements.

Figure 4.17 illustrates a wet pond. A summary of the design guidance given for wet ponds is provided in Table 4.6, and a more detailed discussion of design elements is provided in the following sections.

Figure 4.17: Extended Detention Wet Pond


Sediment Forebay
Concrete Outlet Structure
Natural Stone Faced Inlet
Vegetation for Safety & Aesthetics
Wetland Plantings
Maintenance Access
Submergent Plantings

Drainage Area

Wet ponds require a minimum drainage area of about 5 hectares to sustain the permanent pool, unless there is another source of water, such as a high local groundwater table. The preferred drainage area for wet ponds is ≥ 10 hectares.

Volumetric Sizing

A subwatershed plan will provide guidance with respect to the permanent pool and active storage (extended detention) required. If a subwatershed plan has not been completed, please refer to Chapter 3. The larger of the erosion control active storage and the water quality active storage should be provided. Normally, it is not necessary to provide both types of storage. Where erosion control active storage exceeds 40 m^3/ha, the water quality active storage can be neglected (because of similar drawdown characteristics).

Table 4.6: Wet Ponds – Summary of Design Guidance

Design Element	Design Objective	Minimum Criteria	Preferred Criteria
Drainage Area	Volumetric turnover	5 hectares	≥ 10 hectares
Treatment Volume	Provision of appropriate Level of protection (see Section 3.3.1.1)	As per Table 3.2	1. Permanent Pool volume increased by expected maximum ice volume 2. Active Storage increased from 40 m^3/ha to 25% of total volume
Active Storage Detention	Suspended Solids Settling	24 hrs (12 hrs if in conflict with minimum orifice size)	24 hrs
Forebay	Pre-treatment	Minimum Depth: 1 m Sized to ensure non-erosive velocities leaving forebay Maximum Area: 33% of total Permanent Pool	Minimum Depth: 1.5 m Maximum Volume: 20% of total Permanent Pool
Length-to-Width Ratio	Maximize flow path and minimize short-circuiting potential	Overall: minimum 3:1 (may be accomplished by berms, etc.) Forebay: minimum 2:1	From 4:1 to 5:1
Permanent Pool Depth	Minimize re-suspension, avoid anoxic conditions	Maximum Depth: 3 m Mean Depth: 1 m - 2 m	Maximum Depth: 2.5 m Mean Depth: 1 m - 2 m
Active Storage Depth	Storage/Flow Control	Water Quality and Erosion Control: maximum 1.5 m Total (including quantity control): 2 m	Water Quality and Erosion Control: maximum 1.0 m Total (including quantity control): 2 m
Side slopes	Safety Maximize the functionality of the pond	5:1 for 3 m on either side of the permanent pool Maximum 3:1 elsewhere	7:1 near normal water level plus use of 0.3 m steps 4:1 elsewhere

Table 4.6: Wet Ponds – Summary of Design Guidance (cont'd)

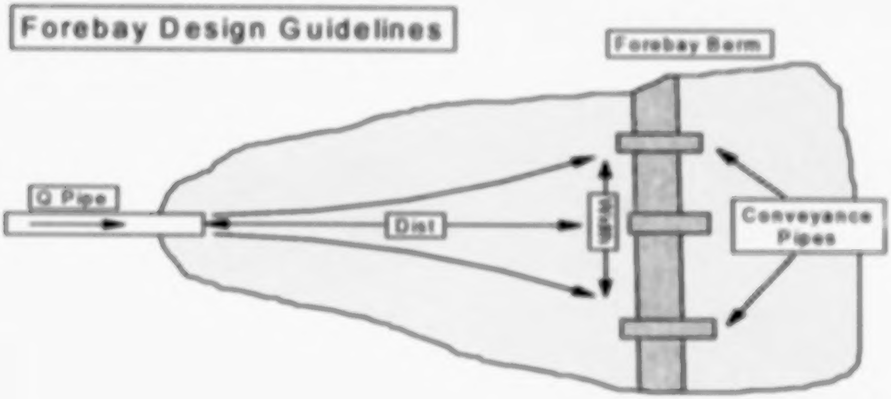
Design Element	Design Objective	Minimum Criteria	Preferred Criteria
Inlet	Avoid clogging/ freezing	Minimum: 450 mm Preferred pipe slope: > 1% If submerged, obvert 150 mm below expected maximum ice depth	
Outlet	Avoid clogging/ freezing	Minimum: 450 mm outlet pipe Reverse sloped pipe should have a minimum diameter of 150 mm Preferred pipe slope: > 1% If orifice control used, 75 mm diameter minimum	Minimum 100 mm orifice
Maintenance Access	Access for backhoes or dredging equipment	Provided to approval of Municipality	Provision of maintenance drawdown pipe
Sediment Drying Area	Sediment removal	While preferable, should only be incorporated into the design when it imposes no additional land requirement	To be provided above maximum water quality water level Drainage returned to Pond
Buffer	Safety	Minimum 7.5 m above maximum water quality/erosion control water level Minimum 3 m above high water level for quantity control	

Sediment Forebay

A sediment forebay (Figure 4.18) facilitates maintenance and improves pollutant removal by trapping larger particles near the inlet of the pond. The forebay should be one of the deeper areas of the pond (at least 1 m) to minimize the potential for re-suspension and to prevent the conveyance of re-suspended material to the pond outlet.

The forebay sizing depends on the inlet configuration, and several calculations can be made to ensure that it is adequately sized.

Figure 4.18: Wet Pond Forebay



1) Settling Calculations

The primary method to calculate the forebay volume and length should be based on settling calculations that determine the distance to settle out a certain size of sediment. The methodology assumes that the flow out of the pond dictates the velocity through the forebay and the rest of the pond. Although this is not strictly correct, it is reasonable for the determination of an appropriate forebay length. Equation 4.5 defines the appropriate forebay length for a given settling velocity and hence, the particle size to be trapped in the forebay.

$$\text{Dist} = \sqrt{\frac{r Q_p}{V_s}}$$

Equation 4.5: Forebay Settling Length

where Dist = forebay length (m)
r = length-to-width ratio of forebay
Q_p = peak flow rate from the pond during design quality storm
V_s = settling velocity (dependent on desired particle size to settle). It is recommended that a value of 0.00[illegible] m/s be used in most cases.

In all instances the forebay should not exceed one-third of the pond surface area.

2) Dispersion Length

The dispersion length refers to the length of fluid required to slow a jet discharge (i.e., pipe flow). A check can be made on the forebay length given by the settling calculation (Equation 4.5) to ensure that there is adequate dispersion. Equation 4.6 provides a simple guideline for the length of dispersion required to dissipate flows from the inlet pipe. It is recommended that the forebay length is such that a fluid jet will disperse to a velocity of ≤ 0.5 m/s at the forebay berm.

The dispersion length is usually smaller than the settling length unless there is a large upstream urban drainage area (e.g., 100 ha) or the pond is subject to large inflows (i.e., a combined quantity and quality facility). In cases where a combined facility is designed, the dispersion length should be calculated for the pipe design capacity unless the pipe is designed for storms larger than a 10 year return period. In cases where the pipe conveys flows in excess of a 10 year storm, the dispersion length should be calculated for 10 year flows. In all cases, the forebay length should be greater than or equal to the larger of the lengths given by Equation 4.5 and Equation 4.6.

$$\text{Dist} = \frac{8Q}{dV_f}$$

Equation 4.6: Dispersion Length

where Dist = length of dispersion (m)
Q = inlet flowrate (m^3/s)
d = depth of the permanent pool in the forebay (m)
V_f = desired velocity in the forebay (m/s)

The depth of the permanent pool in the forebay in Equation 4.6 reflects the deep section (> 1 m) of the forebay required to minimize re-suspension and scour. A guideline for the minimum bottom width of this deep zone is given by Equation 4.7:

$$\text{Width} = \frac{\text{Dist}}{8}$$

Equation 4.7: Minimum Forebay Deep Zone Bottom Width

Generally, the total width of the forebay should provide a length-to-width ratio ≥ 2:1 if a single inlet is proposed for the pond to maximize effective storage (i.e., minimize dead zones).

Although Equation 4.6 provides the length of forebay that ensures a certain velocity in the discharge jet at the end of the forebay, a check should be made using the entire forebay cross-sectional area to ensure that the average velocity in the forebay is less than, or equal to, 0.15 m/s which is empirically recognized as the maximum permissible velocity before which erosion will occur in a channel.

The design flow rate in Equation 4.6 is the peak flowrate of the water quality storm. If this value is not known (e.g., the subwatershed plan specifies the pond sizing based on continuous simulation) it can be approximated using either standard design event modelling practices with a 4 hour Chicago distribution of a 25 mm storm, or using the Rational Method (Equation 4.8) with an intensity given by Equation 4.9.

$$Q = \frac{C\,i\,A}{360}$$ **Equation 4.8: Rational Method**

where Q = peak flow rate (m^3/s)
C = runoff coefficient
i = rainfall intensity (mm/h)
A = drainage area (ha)

$$i = 43\,C + 5.9$$ **Equation 4.9: 25 mm Storm Intensity**

where i = rainfall intensity (mm/h)
C = runoff coefficient

3) Clearout Frequency

A check on the permanent pool volume contained in the forebay can be made by estimating the accumulation of sediments in the forebay. A conservative estimate would be to assume the maximum facility removal efficiency in the forebay and to ensure that the forebay volume is equal to, or greater than, 10 years of sediment accumulation. Values of sediment loading/ accumulation per hectare of contributing drainage area are provided in Section 6.4 (Table 6.3) based on the upstream catchment imperviousness.

Forebay Berm

The forebay should be separated from the rest of the pond by an earthen berm. The berm can be submerged slightly below the permanent pool or it can extend into the extended detention portion of the pond. Pipes can be installed in the berm as either the primary conveyance system from the forebay to the pond, or as a secondary conveyance system to supplement flows over a submerged berm. In either case, flow calculations should be made to ensure that the berm does not provide a flow restriction which would cause the entire forebay (not just the berm) to overflow under design conditions. The calculations should account for the potential ice thickness over the berm.

The inverts of any conveyance pipes installed in the berm should be set at least 0.6 m above the bottom of the forebay. This will prevent the siphoning of settled material from the bottom of the forebay into the rest of the pond. A maintenance pipe should also be installed in the berm to draw down the forebay for maintenance purposes. If only the forebay is drawn down during maintenance (i.e., maintenance pipe connects to the outlet directly and or the forebay will be pumped out) the forebay berm must be designed as a small dam since the rest of the pond will not be drained.

If a submerged berm is used, the berm height should be 0.15 metres - 0.30 metres below the permanent pool elevation. A submerged berm provides additional safety benefits (the public is not tempted to walk on the berm) and may be planted with emergent vegetation to promote filtration of water as it passes over the berm.

Detention Time

A detention time of 24 hours should be targeted in all instances, unless the outlet is susceptible to clogging due to its small size (i.e., drainage areas < 8 ha). If the outlet may be prone to clogging, the detention time can be reduced to a minimum of 12 hours. The detention time is approximated by the drawdown time.

The drawdown time in the pond can be estimated using Equation 4.10. Equation 4.10 is the classic falling head orifice equation which assumes a constant pond surface area. This assumption is generally not valid, and a more accurate estimation can be made if Equation 4.10 is solved as a differential equation. This is easily done if the relationship between pond surface area and pond depth is approximated using a linear regression.

$$t = \frac{2A_p}{CA_o(2g)^{0.5}}\left(h_1^{0.5} - h_2^{0.5}\right)$$

Equation 4.10: Drawdown Time

or if a relationship between A_p and h is known (i.e., $A = C_2 h + C_3$)

$$t = \frac{0.66\,C_2 h^{1.5} + 2\,C_3 h^{0.5}}{2.75\,A_o}$$

Equation 4.11

where t = drawdown time in seconds
A_p = surface area of the pond (m^2)
C = discharge coefficient (typically 0.63)
A_o = cross-sectional area of the orifice (m^2)
g = gravitational acceleration constant (9.81 m/s^2)
h_1 = starting water elevation above the orifice (m)
h_2 = ending water elevation above the orifice (m)
h = maximum water elevation above the orifice (m)
C_2 = slope coefficient from the area-depth linear regression
C_3 = intercept from the area-depth linear regression

Minimum Orifice Size

The smallest diameter orifice accepted by most municipalities to ensure that clogging does not occur in a stormwater system is 75 mm. The preferred minimum orifice size is 100 mm where the effects of freezing are a concern. It is recommended that this latter size be maintained for exposed outlet designs (i.e., reverse sloped pipes). In instances where a perforated riser outlet is designed, the orifice is protected by the smaller perforations in the riser and a minimum orifice size of 50 mm is acceptable. Where small orifices are required, consideration should be given to providing an overflow outlet which would operate in the event that blockage of the primary orifice occurs.

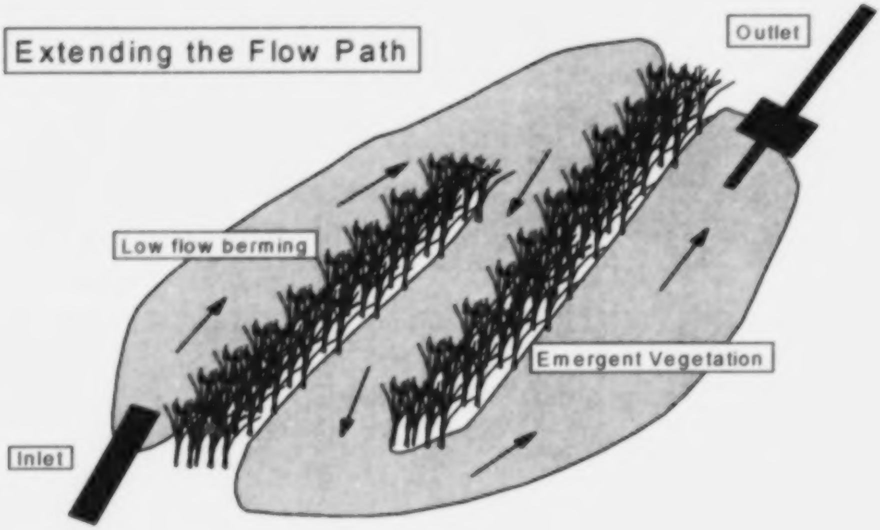
Length:Width Ratio

The flow path through a pond directly influences the overall performance. One of the most common problems associated with first generation pond designs was the construction of the outlet close to the inlet. Another common problem involved having multiple stormwater inlets at opposite ends of the pond based on stormwater servicing convenience. In each case, the effective volume of the facility can be reduced.

Wherever possible, all stormwater servicing should be conveyed to one inlet location at the pond. In order to provide the longest flow path through the pond, the inlet to the pond should be located as far away as possible from the pond outlet. Recognizing that a specific storage volume is required to be provided in the pond, the minimum flow path in a pond is generally described by the length-to-width ratio. A pond with a length-to-width ratio ≥ 3:1 will have an acceptable flow path. Preferred length-to-width ratios range from 4:1 to 5:1.

Berms in the pond to re-direct flows at certain elevations and lengthen the effective flow path are an acceptable design feature. They may improve pond performance by ensuring that short-circuiting cannot occur. If the berms are vegetated, the vegetation will help to filter the stormwater, further enhancing the performance. In some areas of the province, this is called a "serpentine" design (Figure 4.19). The addition of berms, however, will increase the land consumption of ponds.

Figure 4.19: Low Flow Berming



Permanent Pool and Active Storage Depths

The recommended depths for permanent and active storage are based on a variety of issues, including the potential for stratification of the water column and the tolerance of plants to water level fluctuations. In addition, some municipalities specify maximum allowable depths related to safety concerns. While technical guidance is provided below, the specific requirements of the local municipality should be checked.

The average permanent pool depth in a wet pond should be 1 to 2 metres. The maximum depth in a wet pond should be restricted to 3 metres, or preferably 2.5 metres. Although ponds deeper than 3 metres may have some benefits in terms of temperature, deep ponds will become stratified and the reduced oxygen content may create anoxic conditions releasing metals and organics from the pond sediments.

The maximum active storage depth (above the permanent pool) should be limited to 2 metres if the facility incorporates quantity control storage. Active storage depths of water quality or erosion control should be restricted to a maximum of 1.5 metres (preferably 1.0 metre) due to the extended drawdown times of these types of storage. This depth restriction is based on the inability of vegetation commonly planted along the perimeter of ponds to withstand water level fluctuations in excess of 1 to 1.5 m.

A 0.3 m freeboard should be provided above the design high water level.

Planting Strategy

A planting strategy is required for any wet facility to provide shading, aesthetic, safety, bird control, enhanced pollutant removal, and other benefits. Section 4.6.1 describes the effective use of vegetation for these and other goals.

Fencing

The installation of full perimeter chain link fencing may be aesthetically undesirable. However, the use of permanent fencing is left to the discretion of the local municipality because of concerns for liability. Alternatives such as the strategic planting of thorn bearing trees and shrubs such as hawthorn and raspberry have proven very effective barriers. Fencing may be necessary in critical areas such as above headwalls or in other areas with significant changes in grade.

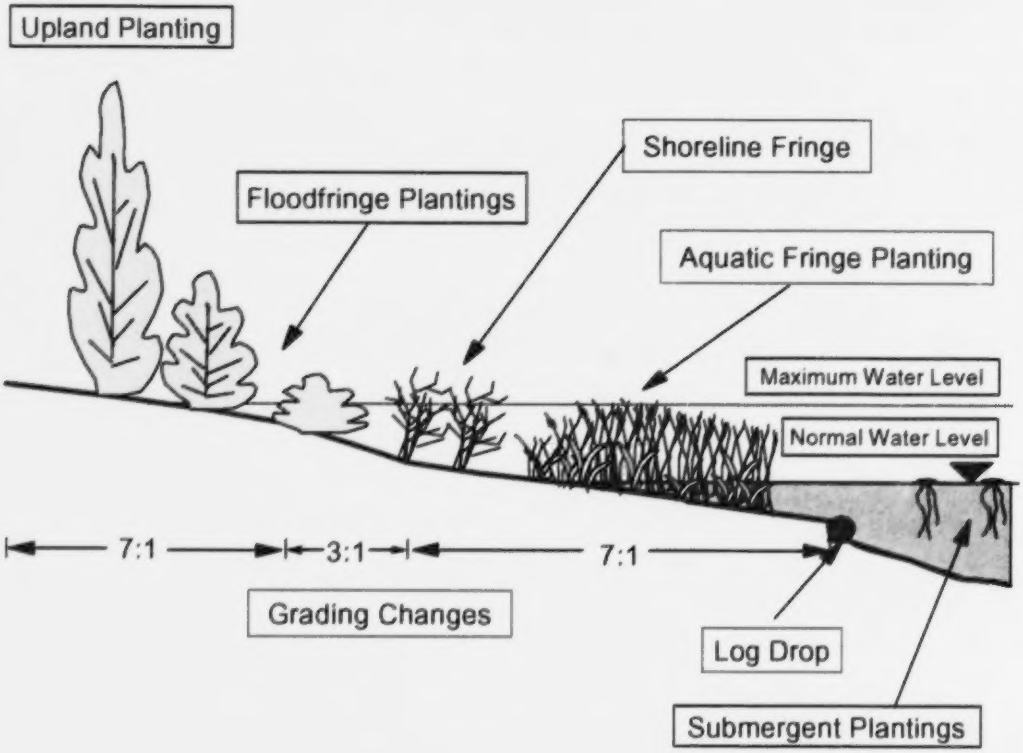
During the design stage, opportunities to incorporate amenities such as trails and seating areas around the SWMP should be explored to encourage safe public access and enhance the value of the facility within the community. Where barriers are required, the use of vegetation should be considered as the preferred option. The erection of temporary fencing may be required to mitigate access until thorny vegetation matures. Once a dense thicket is established, any fencing can be removed and the plant material alone will be sufficient to deter access over the long term.

Signs around the pond indicating the pond's purpose and function also help to inform the public of the potential for water level fluctuations in the pond during storm events.

Grading

The grading and landscaping plan near the pond edges is important to ensure public safety and to maximize the functionality of the pond. Terraced grading (i.e., alternating steep and gentle slopes) is recommended to minimize the potential for any person to fall into the pond (Figure 4.20). Grading is critical at the permanent pool elevation. A minimum slope of 5:1 that extends at least 3 metres on either side of the permanent pool elevation is recommended. At the edge of the gentle grade in the permanent pool, small drops (150 mm - 300 mm) can be incorporated using logs or stones to warn people who gain access to the water that the pond is becoming deeper. Slopes in the extended detention portion of the pond should not exceed 3:1.

Figure 4.20: Wet Pond Grading and Planting Strategy


Upland Planting
Shoreline Fringe
Floodfringe Plantings
Aquatic Fringe Planting
Maximum Water Level
Normal Water Level
7:1
3:1
7:1
Grading Changes
Log Drop
Submergent Plantings

Grading is one of the simplest and most cost-effective tools available to enhance both the appearance and function of ponds, wetlands and hybrid facilities. The grading plan should be generated with the objective of achieving a facility with topography which blends with the surrounding landscape. The abandonment of planar grading and standard side slopes in favour of varied contour grading is fundamental to the creation of a facility that appears to fit within its context. The following principles should be applied to guide the generation of an effective grading plan:

- Design grading to blend with surrounding landform character;
- Employ a range of gradients to achieve objectives related to appearance, water quality improvement, biodiversity and recreational use;
- Design grading with a recognition of the requirements of the desired plant communities, notably soil moisture conditions, and frequency and duration of inundation;
- Configure grading to aid in the concealment of structures, including outfalls, maintenance access routes and weirs;
- Design landforms to increase the potential to establish shade over wet pond areas;
- Utilize islands and varied bathymetry to improve contact time and extend the length of the flow path; and
- Terrace wetlands to extend contact time, improve efficiency and mitigate the potential for short-circuiting.

Grading is a requirement in the construction of all stormwater management facilities, and consequently the implementation of landform grading is a technique that is easily adapted to ponds, wetlands and other types of facilities with minimal implication related to increased cost or complexity. The benefits which can be achieved through the application of this technique, however, are substantial.

Inlet Configuration

The stormwater conveyance system (sewers, grassed swales) should ideally have one discharge location into the wet pond. This requires planning and ongoing interaction between land use planners and municipal engineers to ensure that it is technically feasible and economically efficient. Multiple inlets, although undesirable, may be required because of physical and economic constraints.

Exposed pilot channels (typically rock lined channels which convey stormwater from a pipe outlet to the pond) should be avoided. Monitoring has indicated that water temperature is increased 1°C for every 75 metres of pilot channel (Galli, 1990).

In areas where there is sufficient topographic relief, the inlet can be submerged below the permanent pool. This design has both advantages and drawbacks:

Advantages:

- safety and prevention of vandalism; and
- aesthetics.

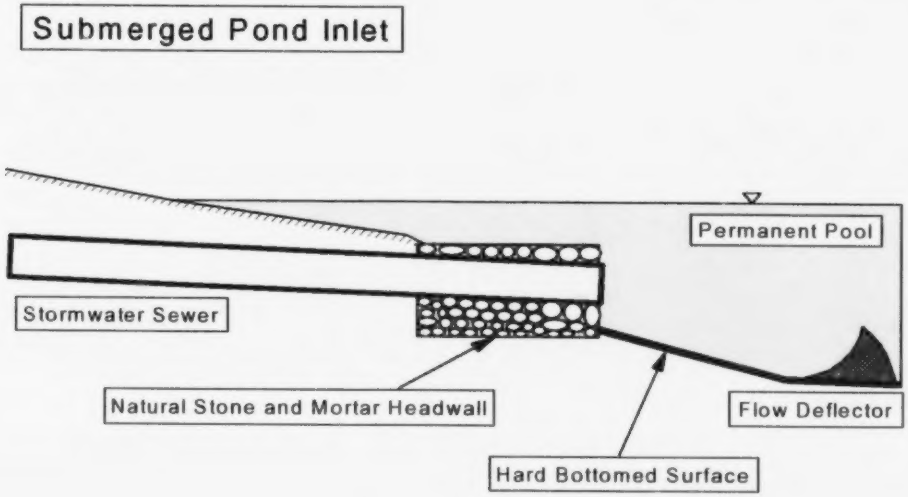
Drawbacks:

- surcharging or backwater effect on the upstream stormwater conveyance system;
- scour/re-suspension of the pond bottom near the inlet;
- clogging of the inlet by sedimentation near the inlet; and
- sediment deposition in the upstream conveyance system.

A design which incorporates a submerged inlet requires a greater level of analysis at the design stage due to these potential drawbacks.

Submerged inlets should not be located at the bottom of the pond unless necessary. If the inlet is located at the pond bottom, a hard-bottomed surface near the inlet pipe is required to ensure that erosion and scour of the pond bottom do not occur. Other enhancements such as dissipation or deflection structures which direct flow away from the pond bottom also help to minimize scour and re-suspension of deposited sediment (Figure 4.21). Submerged inlets for piped systems with a flat grade (< 1%) should be avoided due to the potential for upstream surcharging. (As a rule of thumb only the last 10 metres of pipe should be submerged near the discharge point.)

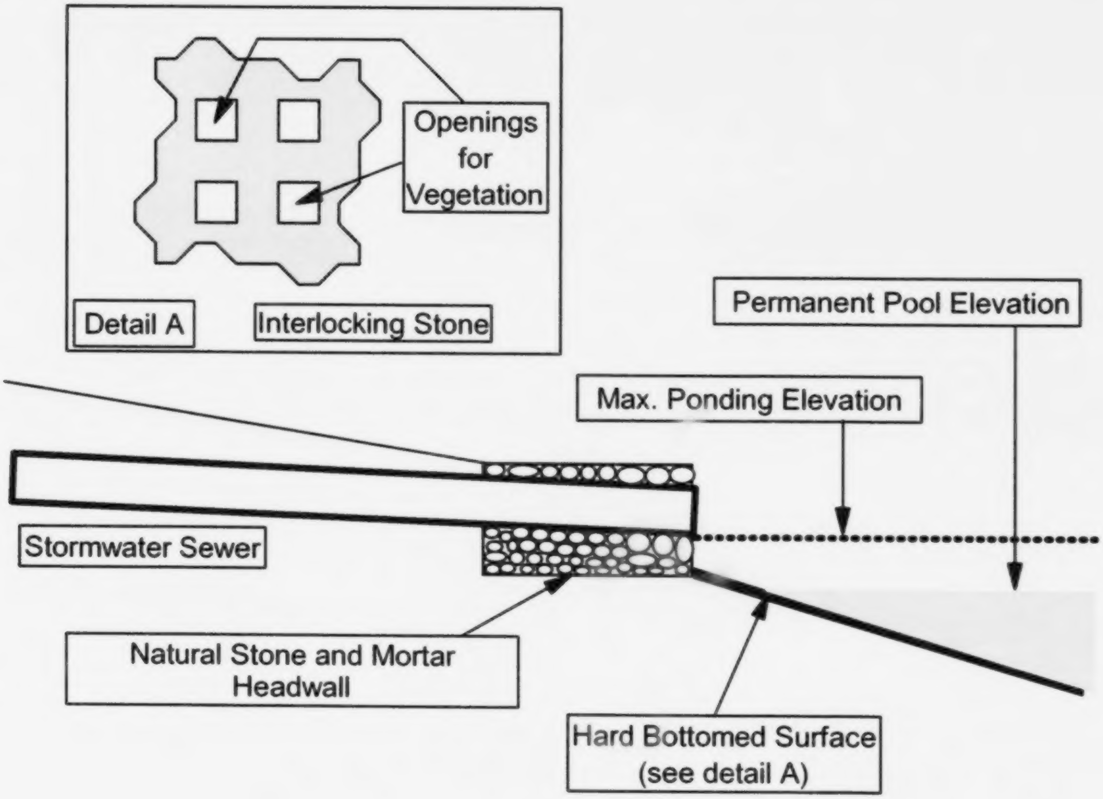
Figure 4.21: Submerged Pond Inlet



The effect of the tailwater condition produced by a partially submerged, or submerged, inlet must be assessed with respect to upstream surcharging during minor system and major system events. Major system events are analysed to determine the requirements for inlet controls on catchbasins. The effect of partial or complete submergence is best evaluated using a dynamic hydraulic routing model such as EXTRAN. A conservative steady state analysis can be made, however, by assuming a constant tailwater elevation equivalent to the maximum design water level in the pond and assessing the upstream surcharge for the peak design event flow.

Non-submerged (i.e., not submerged or partially submerged) inlets are generally easier to design since they do not introduce hydraulic complications into the system and are generally preferred over submerged inlets for the reasons described above (Figure 4.22).

Figure 4.22: Non-submerged Pond Inlet


Openings for Vegetation
Detail A
Interlocking Stone
Permanent Pool Elevation
Max. Ponding Elevation
Stormwater Sewer
Natural Stone and Mortar Headwall
Hard Bottomed Surface (see detail A)

As a result of the raised inlet invert and pond side slopes, the non-submerged inlet will not discharge directly into water. It is important that any erosion potential between the inlet and the permanent pool be addressed. The inlet area should be deep (> 1 metre) to minimize concerns for re-suspension of settled pollutants.

To better integrate the design of the inlet structure within the aesthetics of a natural landscape, consideration should be given to replacing concrete headwalls and wing walls with retaining walls constructed of natural stone and plant material. These biotechnical structures provide the necessary stability and erosion protection while affording the following benefits:

- reduced cost;
- ease of construction;
- concealment of infrastructure; and
- habitat benefits.

The use of stone indigenous to the area helps to blend the facility into its physiographic context. Concrete aprons and chute blocks can be replaced with a plunge pool and planted outlet weir. The pool functions to dissipate energy and moderate velocities which in turn aid in limiting the re-suspension of accumulated sediments in the forebay. Plunge pools should be excavated to a greater depth than required and allowed to fill in and reshape to correspond with flow characteristics. Once this evolution of form has taken place, the plunge pool will maintain itself at the required depth. An outlet weir is used to control the water level in the plunge pool. Plant material is interlaced with riverstone to create a weir that is resistant to breaching and will accumulate trash and other floatables allowing more efficient removal.

Outlet Configuration

The outlet should be located in the pond embankment for ease of operations, maintenance and aesthetics. There are two main designs which are currently accepted for the drawdown of the quality/erosion portion of the pond:

- a reverse sloped outlet pipe; and
- a perforated riser outlet pipe.

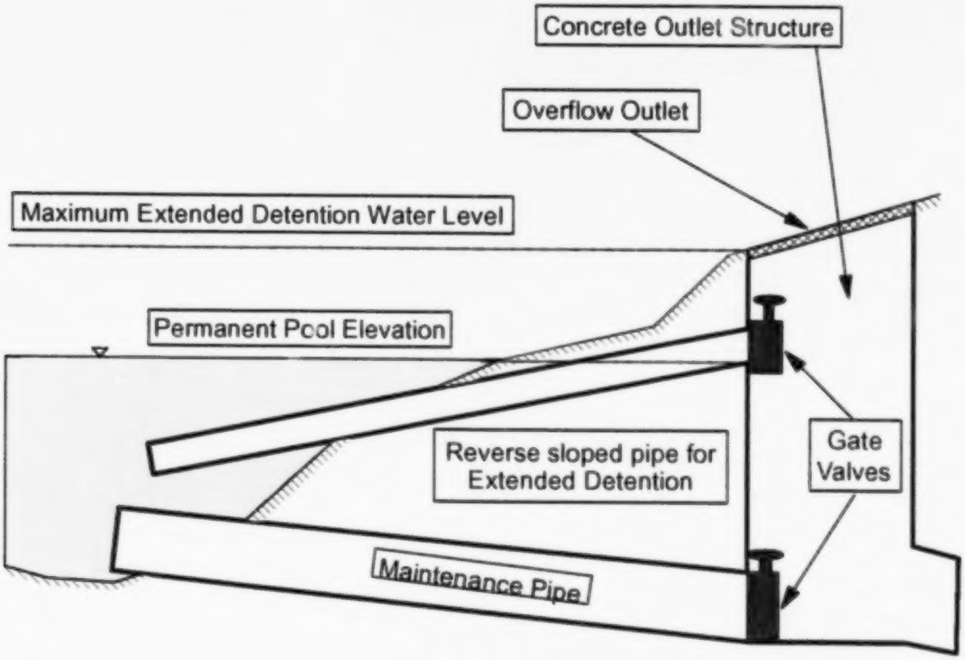
In combined facilities (incorporating quantity control), these types of outlet are usually combined with a weir structure which controls flow at the higher storage levels. Calculations of stage-outflow should account for flow capacity of both the weir and the water quality/erosion control outlet.

Similar techniques to those described for inlets can be employed to integrate outlet structures into the overall landscape of the stormwater management facility. Natural stone and plant material can be used in place of concrete and hard structures to improve aesthetics and achieve other related objectives. Planted weirs are effective in controlling flows at the outlet of sediment forebays. Seepage outlets which are designed to infiltrate water and facilitate the slow release of water to augment base flow in receiving watercourses or wetlands can be constructed utilizing a porous planted weir or hybrid structural/non-structural sand filter systems.

Reverse Sloped Pipe

A reverse sloped pipe (Figure 4.23) is appropriate for ponds with outlet areas ≥ 1 m deep. The reverse sloped pipe is used as the outlet in the water quality/erosion portion of the pond. It should drain to an outlet chamber located in the pond embankment. The outlet chamber can contain openings for flood control detention and overflow protection. It is recommended that a gate valve be attached to the reverse sloped pipe in the outlet chamber. This valve will allow the extended detention drawdown time to be modified to improve pollutant removal if the pond is found to be operating outside of the design criteria.

Figure 4.23: Reverse Sloped Pipe Outlet Configuration


Concrete Outlet Structure
Overflow Outlet
Maximum Extended Detention Water Level
Permanent Pool Elevation
Reverse sloped pipe for Extended Detention
Gate Valves
Maintenance Pipe

A low flow maintenance pipe should be provided to drain the pond for maintenance purposes. The maintenance pipe should also drain to the outlet chamber. It is recommended that the maintenance pipe be sized to provide a 6 hour detention time (6 hours was chosen as a reasonable time period in which to drain the entire pond for maintenance recognizing that the release rate should not affect the downstream receiving waters), and that a gate valve be attached to the end of this pipe in the outlet chamber.

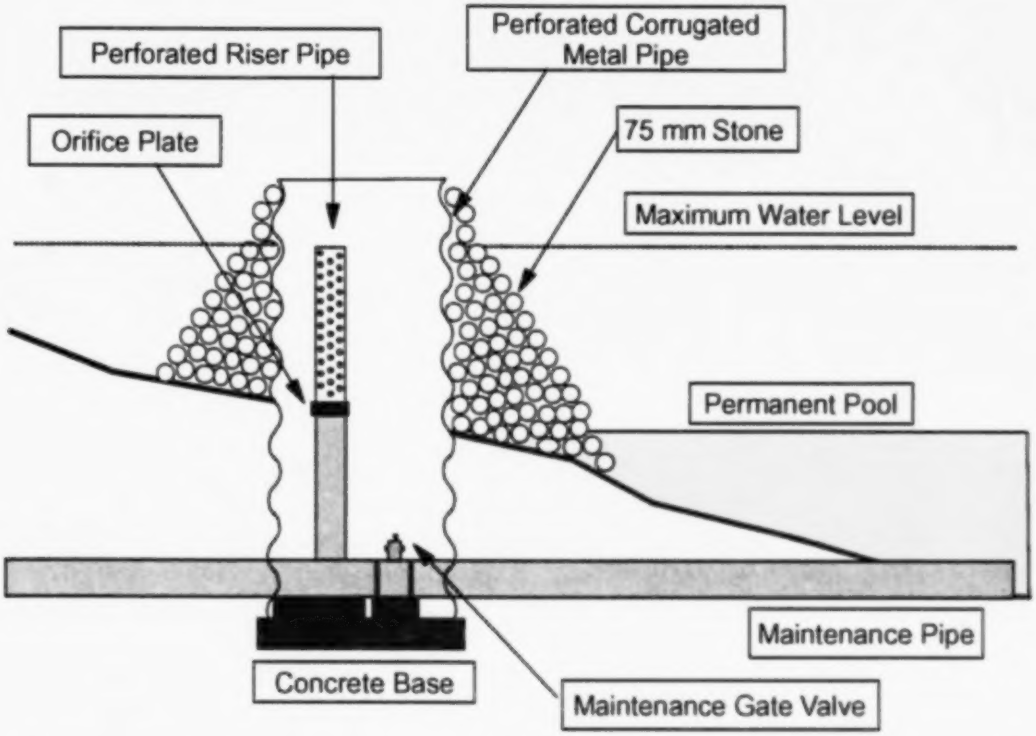
Perforated Riser Pipe

A perforated riser pipe is the traditional outlet pipe that has been used historically throughout Ontario, although its use has diminished in recent years. The riser itself is perforated with holes. Typical hole diameters range from 12 mm to 25 mm.

The flow through the riser is controlled by an orifice plate located at the bottom of the riser structure. The smallest orifice diameter which should be used is 50 mm.

A design which is frequently used in Ontario incorporates a perforated riser pipe surrounded by a corrugated metal pipe standing on its end. Holes (50 mm diameter) are drilled in the metal pipe such that it acts as a riser. Stone is placed around the metal pipe (minimum 75 mm diameter) to act as a further filter. This design is shown in Figure 4.24.

Figure 4.24: Perforated Riser Pipe Pond Outlet Configuration

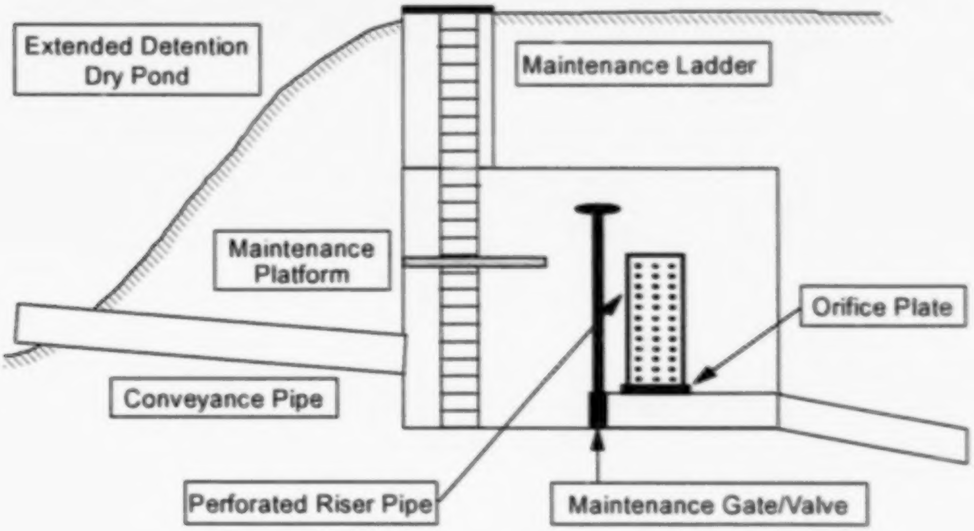

Perforated Riser Pipe
Perforated Corrugated Metal Pipe
Orifice Plate
75 mm Stone
Maximum Water Level
Permanent Pool
Maintenance Pipe
Concrete Base
Maintenance Gate Valve

Although this design is inexpensive, and should be resilient to clogging by suspended solids, there are several drawbacks to keep in mind if this design is chosen:

- If the structure is not located in a chamber in the embankment it will have to be located in the pond itself. This type of outlet will look unnatural and is aesthetically unappealing;
- Corrugated metal pipe which has holes drilled in it will rust resulting in a shorter life span compared to other materials; and
- Since the riser is above the permanent pool it will be more susceptible to clogging by trash.

A similar outlet structure in the embankment (to address aesthetics and maintenance access) is provided in Figure 4.25.

Figure 4.25: Perforated Riser Outlet in Embankment


Extended Detention Dry Pond
Maintenance Ladder
Maintenance Platform
Orifice Plate
Conveyance Pipe
Perforated Riser Pipe
Maintenance Gate/Valve

Water can be conveyed to the chamber by either a positively sloped pipe (> 1%) or a reverse sloped pipe (> 1%). If a positively sloped conveyance pipe is used, it should be larger than 250 mm diameter to minimize the risk of clogging.

The fittings and the riser itself should be constructed of a durable plastic or similar material. Holes should be drilled into the riser (13 mm - 25 mm diameter) along its entire length. The diameter of the pipe, and hence the number of openings, should be sufficient to ensure that the openings do not provide the extended detention control. The riser should be connected to the outlet pipe discharging from the chamber.

In the event that the riser does clog, there should be a maintenance gate or valve in the outlet chamber. A by-pass pipe which routes flows directly to the outlet pipe around the chamber is preferable, but more expensive.

Although the perforated riser outlet design has been used for wet ponds, it is best suited to ponds with a shallow permanent pool (i.e., wetlands) or to dry ponds.

Outlet Channel

Most ponds which discharge directly to a stream will be in close proximity to the receiving waters (the pond will likely be located in the lowest section of the tableland adjacent to the receiving waters). The outlet channel in most of these cases will be short. In cases where the outlet channel is lengthy (i.e., traverses a wide floodplain), natural channel design techniques should be used to ensure that the channel conforms to the natural characteristics of the valleylands. Guidance on natural channel design techniques is provided in "Natural Channel Systems: An Approach to Management and Design" (Ministry of Natural Resources, 1994).

Winter Operation

In areas of the province where ice cover persists into the spring, it is recommended that the volume of the permanent pool be increased by an amount equal to the expected volume of ice cover, as described in Section 4.3.

Because of concerns for winter operation, the minimum diameter of inlet pipes should be 450 mm. A slope of ≥ 1% should be used where possible. Submerged and partially submerged inlets should be avoided. Where a submerged inlet is required, its obvert should be located 150 mm below the expected maximum ice depth.

Submerged outlets (reverse sloped pipe, baffle plate) should be set 150 mm below the expected maximum ice level; reverse sloped pipes should have minimum diameter of 150 mm.

Maintenance Enhancements

The practice of providing a hard-bottomed forebay has met with variable success. The hard-bottom design was originally intended to allow small machines (such as a backhoe) to operate with ease in scraping up sediment in a drained forebay. Experience has shown that the inter-locking blocks (most often used for bottom-hardening) will often be torn up by the scraping operation. Further, equipment such as long-reach backhoes have become more readily available for pond maintenance, and hence, there is less need for equipment entry into the pond. Therefore, unless some special condition warrants it, hardening of the forebay is no longer recommended.

Hardening of portions of the forebay (e.g., at a submerged inlet) to prevent erosion continues to be recommended.

The provision of a sediment drying area is a design enhancement which is desirable. However, the area needed for drying may be relatively large and may not be economically justifiable, especially given the expected frequency of clear-out after the development has stabilized. Further, equipment is available which can deal with high water-content sediments. Hence, a sediment drying area, while preferable, should only be incorporated into the design when it imposes no additional land requirement (such as when a public park abuts a SWMP facility and can be incorporated into the design).

4.6.3 Constructed Wetlands

The constructed wetland is one of the preferred end-of-pipe SWM facilities for water quality enhancement. Wetlands are normally more land-intensive than wet ponds because of their shallower depth (both in the permanent pool and in the active storage zone). They are suitable for providing the storage needed for erosion control purposes, but will generally be limited in their quantity (i.e., flood) control role because of the restrictions on active storage depth.

The benefits of constructed wetlands are similar to wet ponds and include:

- the performance does not depend on soil characteristics;
- the permanent pool minimizes re-suspension;
- the permanent pool minimizes blockage of the outlet;
- the biological removal of pollutants (enhanced nutrient removal) occurs; and
- the permanent pool provides extended settling.

Constructed wetlands also have similar environmental impacts to wet ponds related to increased downstream water temperature which may limit their application in certain areas.

Limited performance monitoring has been conducted for wetland systems in Ontario, and constructed wetlands are the least understood end-of-pipe SWM facilities in terms of their biological impacts and enhancements. Although wetlands have been noted to accumulate total phosphorus, they export ortho-phosphorus (the form of phosphorus which results in algal blooms) and metals such as zinc during the fall as the wetland plants begin to decompose (Novotny, 1983; Martin, 1988; Bayley et al., 1986). These findings have given rise to the harvesting of wetland plant material to prevent the export of pollutants, while others argue that the release of contaminants (namely phosphorus) during the fall has a negligible impact on downstream resources.

Wetlands are used in Ontario for stormwater quality control and in some cases, as biological treatment facilities for other types of effluent. The latter type of facility requires a Schedule C Class EA, under the Municipal Class Environmental Assessment (2000). Although biological treatment occurs in constructed wetlands, physical processes, such as sedimentation and filtering, are the predominant removal mechanisms for most stormwater contaminants. Therefore, the Municipal Class EA requirements for stormwater treatment wetlands are equivalent to those for stormwater treatment ponds and infiltration systems.

Design Guidance

A summary of design guidance is provided in Table 4.7. A more detailed discussion of this guidance is provided in the sections that follow.

Table 4.7: Wetlands – Summary of Design Guidance

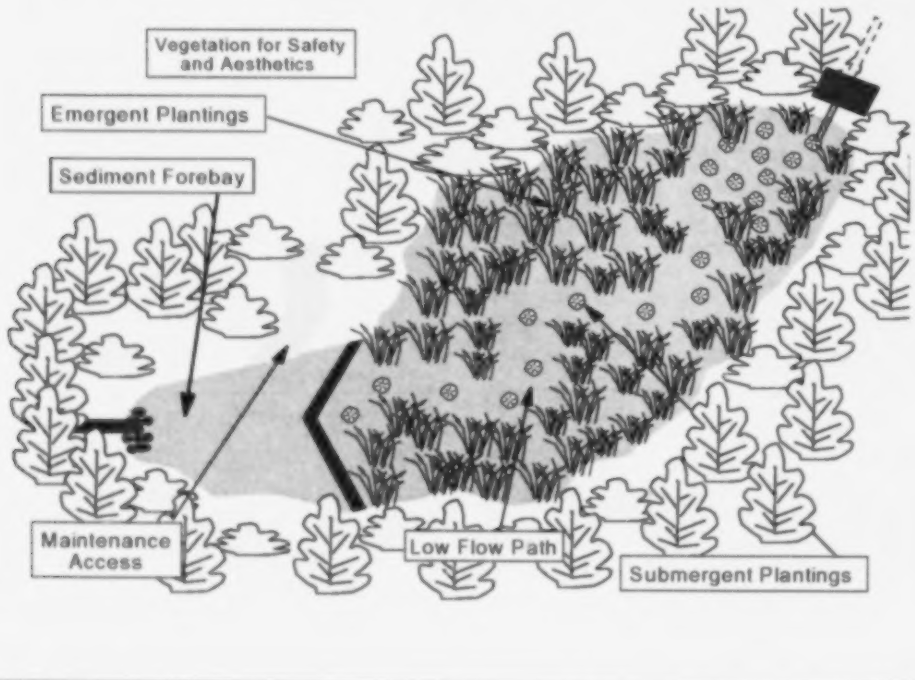
Design Element	Design Objective	Minimum Criteria
Drainage Area	Sustaining Vegetation; Volumetric turnover	5 hectares 10 hectares preferable
Treatment Volume	Provision of appropriate Level of protection (see Section 3.3.1.1)	As per Table 3.2
Active Storage Detention Time	Suspended Solids Settling	24 hrs (12 hrs if in conflict with minimum orifice size)
Forebay	Pre-treatment	Minimum Depth: 1 m Sized to ensure non-erosive velocities leaving forebay Maximum Area: 20% of total Permanent Pool
Length-to-Width Ratio	Maximize flow path and minimize short-circuiting potential	Overall: minimum 3:1 Forebay: minimum 2:1
Permanent Pool Depth	Vegetation requirements, rapid settling	The average permanent pool depth should range from 150 mm to 300 mm
Active Storage Depth	Storage/Flow Control Sustaining Vegetation	Maximum 1.0 m for storms < 10 year event
Side Slopes	Safety	5:1 for 3 m above and below permanent pool Maximum 3:1 elsewhere
Inlet	Avoid clogging/freezing	Minimum 450 mm Preferred pipe slope > 1% If submerged, obvert 0.15 m below expected maximum ice depth

Table 4.7: Wetlands – Summary of Design Guidance (cont'd)

Design Element	Design Objective	Minimum Criteria
Outlet	Avoid clogging/freezing	Minimum: 450 mm outlet pipe Preferred pipe slope > 1% If orifice control used, 75 mm minimum Minimum 100 mm orifice preferable
Maintenance Access	Access for backhoes or dredging equipment	Provided to approval of Municipality Provision of maintenance drawdown pipe preferred
Buffer	Safety	Minimum 7.5 m above maximum water quality/erosion control water level

In many cases the design elements for wetlands are the same as for wet ponds. In such cases, the reader is referred to Section 4.6.2. Figure 4.26 illustrates a constructed wetland.

Figure 4.26: Wetland with Forebay


Vegetation for Safety and Aesthetics
Emergent Plantings
Sediment Forebay
Maintenance Access
Low Flow Path
Submergent Plantings

Drainage Area

Wetlands require a minimum drainage area to sustain the aquatic vegetation and the permanent pool. As a general rule, wetlands should be implemented for drainage areas ≥ 5 hectares, and preferably 10 hectares or more. Smaller drainage areas may be viable when there is a high groundwater table or a source of make-up water.

Volumetric Sizing

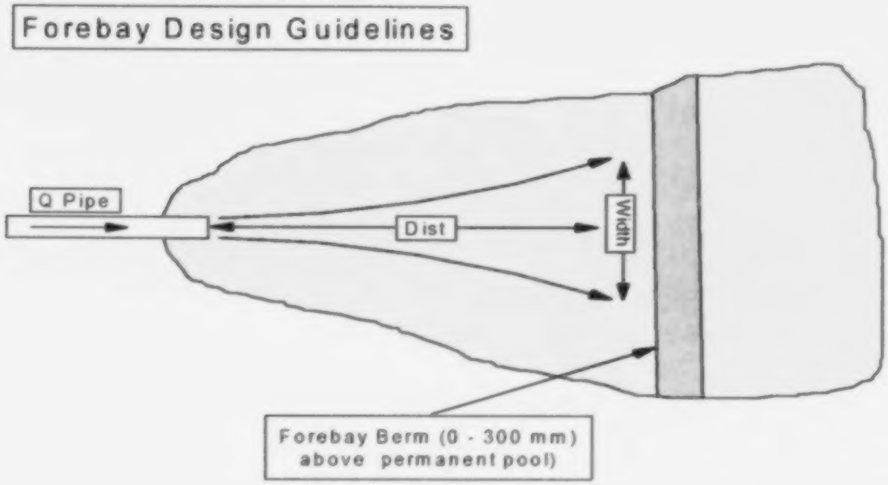
A subwatershed plan will provide guidance with respect to the permanent pool and extended detention storage required. If this guidance is not available, please refer to Chapter 3. Where erosion control active storage exceeds 40 m^3/ha, the water quality active storage can be neglected (due to similar drawdown characteristics).

Active storage for quantity control will not usually be incorporated into a wetland design because of the practical limits imposed by the maximum allowable active storage depth (to protect wetland vegetation).

Sediment Forebay

Sediment forebays (Figure 4.27) improve pollutant removal by trapping larger particles near the inlet of the wetland. A forebay is especially important in a wetland design since restricting maintenance to this area (for the most part) minimizes the need to disturb the wetland vegetation. The forebay should be deep (≥ 1 metre) to minimize the potential for scour and re-suspension. The forebay sizing depends on the inlet configuration, and several methods for sizing are provided in Section 4.6.2. In all instances, the forebay should not exceed one-fifth of the wetland surface area.

Figure 4.27: Wetland Forebay


Forebay Design Guidelines
Q Pipe
Dist
Width
Forebay Berm (0 - 300 mm) above permanent pool)

Forebay Berm

The forebay should be separated from the rest of the wetland by an earthen berm. The berm should be set at the permanent pool elevation or extend into the extended detention portion of the wetland to act as a level spreader during storm events, and to minimize the disruption to the wetland during maintenance of the forebay. Flow calculations should be made under design conditions to ensure that the berm does not provide a flow restriction which would cause the whole forebay to overflow (not just into the wetland) because of the restriction of flow into the wetland resulting from a small berm width.

If a by-pass pipe is proposed to convey high flows around the wetland, a maintenance pipe should be installed in the forebay and connected to the by-pass pipe, if grades permit, to draw down the forebay for maintenance purposes.

The berm height should be set at, or within 300 mm of, the permanent pool elevation in the wetland. The berm should be planted with suitable emergent vegetation to promote the filtration of stormwater as it passes over the berm.

Detention Time

Refer to Section 4.6.2.

Minimum Orifice Size

Refer to Section 4.6.2.

Length:Width Ratio

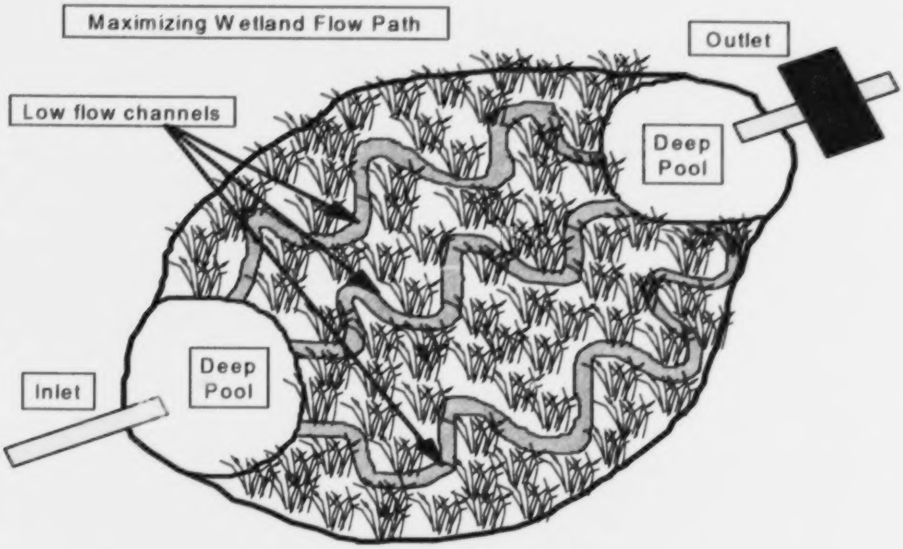
The flow path through a wetland is important to the overall performance of this SWMP. In contrast to the wet pond, however, the flow path in a wetland is mostly dependent on the plantings and grading within the wetland due to the shallow depth of the permanent pool. Although a length-to-width ratio of 3:1 is recommended in stormwater wetlands, it should be measured based on the flow path of low flows through the wetland rather than the overall dimensions of the wetland. Low flow paths should be created through the wetland to ensure that short-circuiting does not occur and that the flow path through the wetland is maximized during small events. Figure 4.28 illustrates the concept of maximizing the length of low flow paths.

Deep zones perpendicular to the direction of flow have also been suggested as a means of defeating short-circuiting. These zones collect and redistribute water, and also provide additional storage volume.

Permanent Pool and Active Storage Depths

The average permanent pool depth in a wetland should range from 150 mm to 300 mm. Inlet and outlet areas should be deeper ($\geq$ 1.0 m) to minimize the re-suspension and discharge of settled pollutants from the facility. The maximum depth in the inlet and outlet areas should be restricted to 3 metres. The wetland design may incorporate deeper pools scattered throughout the wetland area. The deeper water areas will be mainly open water since they will be too deep to sustain emergent vegetation. As such, the deep areas in wetlands should be limited to 25% (Livingston, 1990) of the total surface area to ensure that the majority of the wetland sustains emergent vegetation.

Figure 4.28: Maximizing Wetland Flow Path



The maximum active storage depth should be limited to 1.0 metre. The depth restriction in the extended detention storage portion of the wetland is related to the planting strategy, since some plant species cannot withstand water level fluctuations in excess of 1 metre. Although the depth of 1 metre is appropriate as a generic value for most wetland designs, the depth of the extended detention storage should be based on the planting strategy that is chosen for the wetland. As such, an aquatic biologist should be consulted for the desired extended detention design depth based on the proposed planting strategy.

Where quantity control is provided, greater depths may be permitted for infrequent events (> 10 year return period).

Planting Strategy
Refer to Section 4.6.1.

Fencing
Refer to Section 4.6.2

Grading

By the nature of the limited allowable permanent pool and extended detention depths, grading in a wetland should be reasonably flat. The side slopes near the permanent pool should be 5:1 or flatter. Slopes in the extended detention portion of the wetland should not exceed 3:1. Terraced grading (Figure 4.20) is recommended to minimize the potential for any person to fall into the wetland.

Grading should be designed to replicate natural landform with varied slopes and gradients. Undulating shoreline configurations are also an effective means to integrate the wetland into the landscape as a feature which is natural in appearance.

Inlet Configuration

The stormwater conveyance system (sewers, grassed swales) should ideally have one discharge location into the wetland. This requires planning and ongoing interaction between land use planners and municipal engineers to ensure that it is economically efficient and feasible to drain the tributary area to one inlet location.

Exposed pilot channels (typically rock lined channels which convey stormwater from a pipe outlet to the wetland) should be avoided. Monitoring has indicated that water temperature is increased by 1°C for every 75 metres of pilot channel (Galli, 1990).

Inlets to wetlands will normally be designed to be non-submerged because of the shallow depth of the permanent pool (even if the forebay is deeper, winter freezing of the downstream shallow areas will normally preclude the use of a submerged inlet). Non-submerged inlets are generally easier to design since they do not introduce hydraulic complications into the system. The invert of the inlet pipe is set at the maximum design water level in the wetland (assuming no flow splitter (Section 4.7) upstream).

As a result of the raised inlet invert and wetland side slopes, the unsubmerged inlet will not discharge directly into water. It is important that the erosion potential between the inlet and the permanent pool be addressed in this design. The use of environmental stone/blocks (inter-locking blocks with large openings to allow vegetative growth in between the blocks) in this area is recommended (Figure 4.22) since they will minimize the erosion potential.

Outlet Configuration

The outlet configuration options for a wetland are similar to those for a wet pond (refer to Section 4.6.2). In general, reverse sloped pipe configurations are recommended when the design incorporates a deep pool at the outlet. A perforated pipe riser is appropriate where a deep pool is not provided.

Winter Operation
During the winter period, much of a wetland's permanent pool volume will be frozen. The wetland will therefore behave in a manner similar to a dry pond, with its active storage component providing the only treatment. The design volumes provided in Table 3.2 ensure a substantial active storage volume relative to the permanent pool volume.

There are limited options for enhancing winter/spring performance in a shallow wetland. The forebay volume can be increased by the estimated volume of ice cover (see Section 4.3.1 for calculations of ice depth). Precautions can be taken to guard against freezing of pipes (Section 4.3.2). Where water quality control is desired during the winter/spring season, a hybrid wet pond/wetland design is often recommended (Section 4.6.4).

Maintenance Enhancements
Refer to Section 4.6.2.

4.6.4 Hybrid Wet Pond/Wetland Systems

Hybrid wet pond/wetland systems consist simply of a wet pond element and a wetland element, connected in series. The system provides for the deep water component which will be least impacted by winter/spring conditions and the wetland component which provides enhanced biological removal during the summer months. In terms of land requirements, it falls between the amounts needed for wet ponds and wetlands.

Hybrid systems present a more diverse range of opportunities to achieve recreational, aesthetic and ecological objectives in the context of the open space system within a new community since they afford greater design flexibility and a diversity of landscape elements.

The design of a hybrid system should be based on the guidance provided for each element (i.e., wet ponds (Section 4.6.2) and wetlands (Section 4.6.3)), with the following clarifications:

- Volumetric sizing of the permanent pool should be based on the Hybrid Wet Pond/Wetland SWMP type in Table 3.2. This assumes that the wet pond comprises 50% of the total permanent pool volume;

- A forebay is required for the wet pond (based on the size of the wet pond, not the entire system) but is not required for the wetland (the wet pond serves this purpose);

- Active storage depth restrictions for wetlands apply to the entire system, unless a terraced, overflow configuration is adopted;

- Detention time for the entire system should be targeted at 24 hours; and

- Length-to-width ratio for the wet pond element may be reduced to 2 to 1, although a higher ratio is encouraged.

4.6.5 Dry Ponds

Dry ponds have no permanent pool of water. As such, while they can be effectively used for erosion control and flood control, the removal of stormwater contaminants in these facilities is purely a function of the detention time in the pond. For a 24 hour retention period, this normally means a lower contaminant removal (the inter-event settling time does not exist). Modelling studies (Perreault et al., Adams, 1996) have indicated that substantial improvement can be made in removal efficiency if a 48 hour detention time can be employed. While achieving this for smaller drainage areas can be difficult (because of orifice size considerations), the use of dry ponds in larger catchments (especially in retrofit situations) may have greater potential than had previously been thought. There are no documented performance monitoring data for dry ponds with longer detention times, however, and re-suspension of settled material remains a concern. As such, the use of dry ponds (for water quality control) remains largely restricted to retrofits, where temperature is an overriding concern, and situations where other more effective SWMP types are infeasible. Dry ponds may be used as part of an overall treatment train approach.

Design Guidance

Figure 4.29 illustrates an extended detention dry pond. A summary of design guidance is provided in Table 4.8. A more detailed discussion of this guidance is provided in the sections that follow. Some of the design elements for dry ponds are the same as for wet ponds. In such cases, the reader is referred to Section 4.6.2. The design guidance provided is for continuous flow-through facilities, the most common form of dry pond. Dry ponds using real-time control or batch operation have been implemented (primarily in the Ottawa area), but most municipalities discourage their use.

Figure 4.29: Extended Detention Dry Pond with Forebay

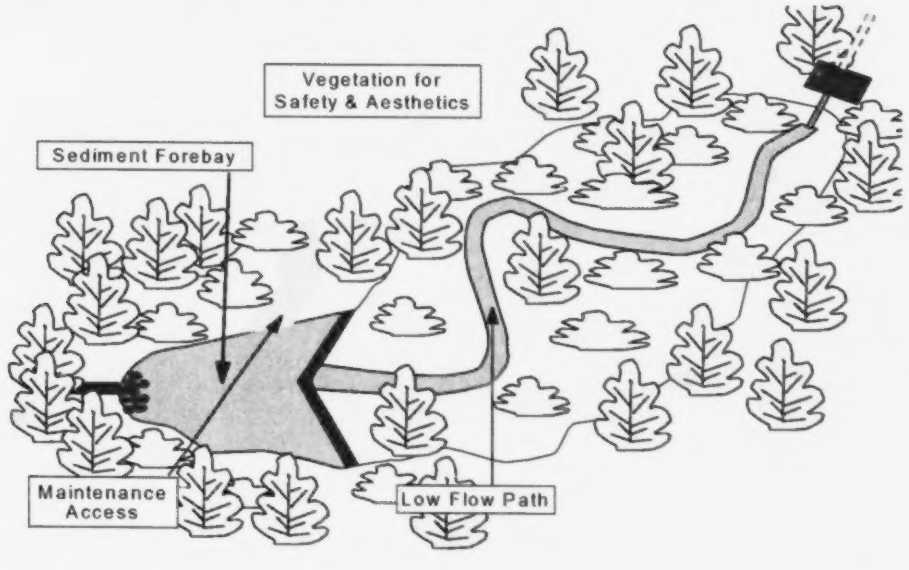


Table 4.8: Dry Ponds (Continuous Flow) – Summary of Design Guidance

Design Element	Design Objective	Minimum Criteria	Desired Criteria
Drainage Area	Minimum Orifice Size	5 hectares	10 hectares
Treatment Volume	Provision of appropriate Level of protection (see Section 3.3.1.1)	As per Table 3.2	
Active Storage Detention Time	Suspended Solids Settling	24 hrs (12 hrs if in conflict with minimum orifice size)	48 hrs
Forebay	Pre-treatment	Minimum Depth: 1 m Sized to ensure non-erosive velocities leaving forebay	Minimum Depth: 1.5 m
Length-to-width Ratio	Maximize flow path and minimize short-circuiting potential	3:1 (may be accomplished by berms, etc.)	From 4:1 to 5:1
Depth	Safety	Maximum Depth: 3 m Mean Depth: 1 - 2 m	Maximum Depth: 2 m Mean Depth: 1 - 2 m
Side Slopes	Safety	Grading of the side slopes terraced with an average slope of 4:1 or flatter	
Inlet	Avoid clogging/freezing	Minimum: 450 mm	Pipe slope > 1%
Outlet	Avoid clogging/freezing	Minimum: 450 mm outlet pipe If orifice control used, 75 mm minimum (unless protected by a riser)	Pipe slope > 1% Minimum 100 mm orifice
Maintenance Access	Access for backhoes or bobcats	Provided to approval of Municipality	Provision of maintenance drawdown pipe in forebay
Buffer	Safety	Minimum 3 m above maximum water level	

Drainage Area

As a general rule, dry ponds should be implemented for drainage areas ≥ 5 hectares. This area requirement is purely a function of the outlet sizing to ensure that the outlet does not become clogged. Smaller drainage areas may be considered (subject to minimum orifice sizing and, hence, reduced detention times) if the dry pond is used as part of an effective treatment train approach (e.g., more than just roof leader discharge to ground).

Volumetric Sizing

A subwatershed plan, or alternatively Chapter 3, will provide guidance with respect to sizing. The larger of the erosion control active storage and the water quality active storage should be provided. Normally (unless 48 hour water quality detention is proposed), it is not necessary to provide both types of storage (due to similar drawdown characteristics).

Dry ponds are often used for flood control. The requirements for erosion control active storage will be dependent on local conditions and policies, as described in Chapter 3.

Sediment Forebay

A sediment forebay facilitates maintenance and improves pollutant removal by trapping larger particles near the inlet of the pond. The forebay should include a deep permanent pool (> 1 metre) to minimize the potential for scour and re-suspension. The forebay sizing depends on the inlet configuration, and several methods for sizing are provided in Section 4.6.2.

Forebay Berm

The forebay should be separated from the rest of the pond by an earthen berm. The berm should be designed as a small dam since the downstream section of the pond will be dry. A weir should be designed at the top of the berm to convey flows to the downstream section of the pond during storm events.

A maintenance pipe should be installed in the berm to allow the forebay to be drawn down for cleaning. This pipe would be opened and closed by a valve located on the upstream end of the pipe. Under normal operation, the valve would be closed such that the only means of conveyance would be the weir flow over the forebay berm. Flow calculations should be made to ensure that the berm does not provide a flow restriction which would cause the entire forebay to overflow (not just over berm) under design conditions because the berm does not provide adequate conveyance capacity. During maintenance periods, the valve would be opened allowing the forebay to be drained.

The berm should be planted with emergent vegetation to promote filtration of water as it passes over the berm.

Detention Time

A minimum detention time of 24 hours should be targeted in all instances, unless the potential for clogging the outlet is high. Where possible, a detention time of 48 hours should be employed to improve suspended solids removal.

In cases where the outlet is susceptible to clogging (i.e., drainage areas < 8 ha – see minimum orifice size), the detention time can be reduced to a minimum of 12 hours. The drawdown time in the pond can be estimated using Equation 4.10 (Section 4.6.2).

Minimum Orifice Size
Refer to Section 4.6.2.

Length:Width Ratio
Refer to Section 4.6.2.

Active Storage Depth
The active storage depth should be limited to 2 to 3 metres. This maximum applies to all extended detention objectives (i.e., water quality, erosion and flood control). Depending upon the proposed planting strategy, these maximum depths may be reduced to 1 to 1.5 metres. It is anticipated, however, that the dry pond will not be actively planted in the extended detention portion due to harsh growing conditions (frequent wetting/drying).

Planting Strategy
The planting strategy for a dry pond is less aggressive (i.e., fewer species and reduced planting intensity) than that for a wet pond. Plantings can be divided into three zones based on the soil moisture regime (frequency of inundation):

- extended detention area;
- flood fringe area (if the facility is a combined quality/quantity SWMP); and
- upland area.

Please refer to Section 4.6.1 for details. The growing conditions in the extended detention area for a dry pond are harsher than in the corresponding area of a wet pond since there is no influence from a permanent pool. Consequently, this area requires close attention to ensure that desirable plants become established.

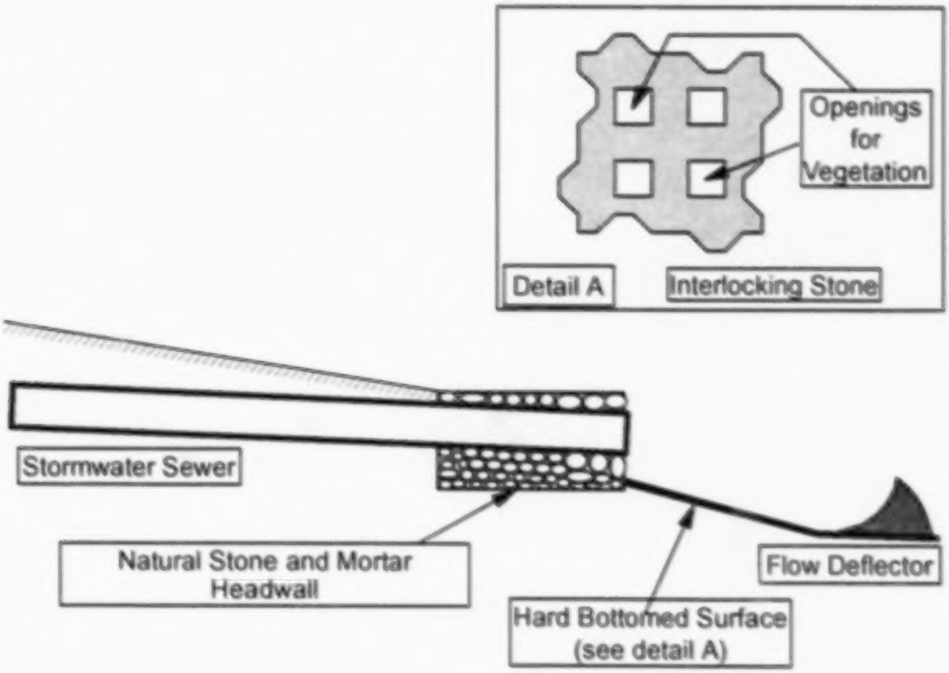
Fencing
Refer to Section 4.6.2.

Grading
The grading in a dry pond is less critical than a wet pond since there is no permanent pool. Since water may be present in these facilities for 24 to 48 hours, it is recommended that the grading of the pond side slopes be terraced with an average slope of 4:1 or flatter.

Inlet Configuration
The stormwater conveyance system (sewers, grassed swales) should minimize the number of discharge locations into the pond. The invert of the inlet pipe is set at the maximum design water

level in the pond (assuming no flow splitter (Section 4.7) upstream). The use of environmental stone/blocks (inter-locking blocks with large openings to allow vegetative growth between the blocks) at the inlet is recommended to minimize the erosion potential (Figure 4.30). A flow deflector or energy dissipation blocks can be used to reduce the potential for scour of previously settled pollutants from the pond bottom.

Figure 4.30: Dry Pond Inlet


Openings for Vegetation
Detail A
Interlocking Stone
Stormwater Sewer
Natural Stone and Mortar Headwall
Flow Deflector
Hard Bottomed Surface (see detail A)

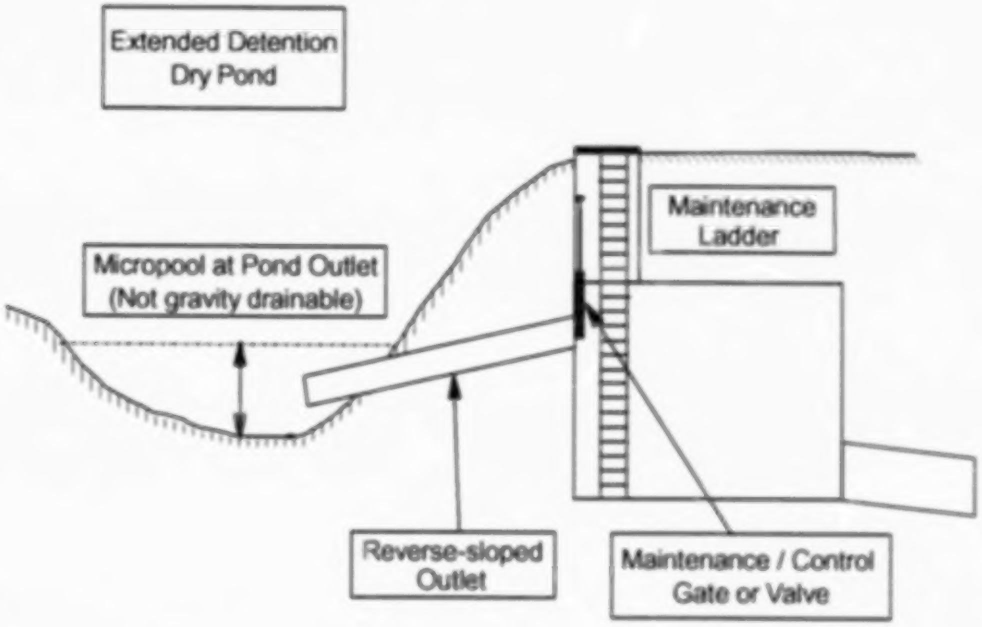
Outlet Configuration

There are numerous outlet configurations possible for dry ponds. The outlet should be located in the pond embankment wherever possible for ease of maintenance and aesthetics. To illustrate typical designs, two configurations are presented: perforated riser and reverse sloped pipe.

A perforated riser pipe can be used as the outlet in the extended detention portion of the pond. As indicated in Section 4.6.2, a perforated riser pipe surrounded by a perforated corrugated metal pipe and 75 mm diameter stones can used. This outlet design is shown in Figure 4.24.

A second type of outlet configuration utilizes a reverse sloped pipe (Figure 4.31). In this design, a section of the pond near the outlet will not be gravity drained due to the reverse slope of the pipe. A portable pump must be used to drain this portion of the pond for maintenance. The use of under-drains (i.e., tile drains) under ponds has not been historically effective in draining these facilities and is not recommended.

Figure 4.31: Dry Pond Reverse Sloped Outlet Pipe


Extended Detention Dry Pond
Maintenance Ladder
Micropool at Pond Outlet (Not gravity drainable)
Reverse-sloped Outlet
Maintenance / Control Gate or Valve

Experience with reverse sloped pipes indicates that they are resilient to clogging (Schueler, 1992). As such, the riser pipe and orifice connection are replaced by a gate valve or sluice gate on the reverse sloped pipe at the outlet chamber. If a valve is used, a gate valve is preferable to a globe valve given the size of valve required, and hence cost. This gate/valve will allow the manipulation of the outlet to achieve the desired settling characteristics in the field.

In addition, since the detention control is located on the inlet pipe to the chamber, rather than the outlet pipe from the chamber, the chamber can be used as a flood control outlet or emergency overflow outlet. For example, the inlet chamber can have a grated top, or weir openings along its side adjacent to the pond to provide further water management control.

Winter Operation

Dry ponds are normally the least affected by winter/spring conditions because there is no permanent pool and the SWMP is not dependent on infiltration. Precautions can be taken to guard against freezing of pipes and orifices, and a by-pass (flow splitter) can be employed to limit inflow (and hence scouring of accumulated sediment) by major spring storms. Otherwise, dry ponds do not require special consideration of winter conditions.

Performance Enhancement

The performance of a dry pond can be enhanced through the provision of a micropool at the outlet. The micropool is typically relatively shallow and undrained. Its purpose is to concentrate finer sediment and reduce re-suspension. The micropool is normally planted with hardy wetland species such as cattail.

4.6.6 Infiltration Basins

Infiltration basins are above-ground pond systems which are constructed in highly pervious soils. Water infiltrates into the basin and either recharges the groundwater system or is collected by an underground perforated pipe network and discharged to a downstream outlet.

Design Guidance

There is limited practical experience with infiltration basins in Ontario. A summary of design guidance is provided in Table 4.9, based on experience in other jurisdictions. A more detailed discussion of this guidance is provided in the sections that follow. Figure 4.32 provides an illustration of an infiltration basin.

Figure 4.32: Infiltration Basin

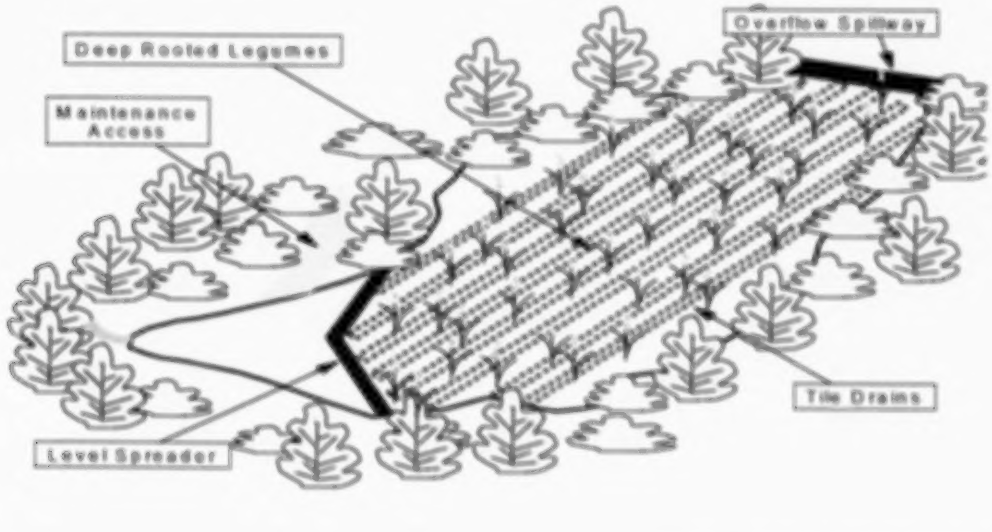


Table 4.9: Infiltration Basins – Summary of Design Guidance

Design Element	Design Objective	Minimum Criteria
Drainage Area	Infiltration	< 5 hectares
Treatment Volume	Provision of appropriate Level of protection (see Section 3.3.1.1)	As per Table 3.2
Percolation Rate	Infiltration	≥ 60 mm/hr
Depth to Water Table	Infiltration	> 1.0 m
Depth to Bedrock	Infiltration	> 1.0 m
Length-to-width Ratio	Spread inflow	3:1 preferred
Storage Depth	Prevent Compaction	< 0.6 m
Pre-treatment	Longevity Groundwater protection	Required Redundancy (more than one device) preferred
By-pass	Winter/spring operation	Required
Maintenance Access	Access for light discing equipment	Provided to approval of municipality
Landscaping Plan	Enhance Infiltration Increase porosity	Grasses, deep rooted legumes

Drainage Area

Infiltration basins should be implemented for small drainage areas (< 5 ha). Although infiltration basins were originally designed to accommodate larger drainage areas, the monitoring which has been undertaken to date indicates that large scale infiltration is not feasible.

Land Use

Infiltration basins are suitable for residential land uses. They are not recommended for industrial and commercial land uses where there is a high potential for groundwater contamination from chemical spills and maintenance (salting/sanding) activities.

Soils

Infiltration basins are not suitable where the native soil has a percolation rate of less than 60 mm/h. Typical percolation rates and soil types are provided in Table 4.4. Table 4.4 should be used as a screening tool to determine if a site may be suitable for an infiltration basin. If a site is acceptable based on the screening process, in situ percolation rates should be determined by a qualified soils specialist or hydrogeologist.

In areas where the soils are marginally acceptable, perforated pipes can be implemented to augment the drainage. Perforated pipes (100 mm diameter) which traverse the entire length of the basin should be spaced a maximum of 1.2 m apart and approximately 300 mm - 600 mm below the ground surface. The perforated pipes should drain to a collection pipe which discharges to an outlet pipe/structure.

Water Table Depth

The seasonally high water table should be more than 1 m below the bottom of the infiltration basin.

Bedrock Depth

The bedrock should be more than 1 m below the bottom of the infiltration basin.

Volumetric Sizing

In areas with a subwatershed plan, the plan will provide guidance with respect to sizing, otherwise, water quality treatment volumes are provided in Table 3.2. The larger of the erosion control active storage and the water quality active storage should be provided.

It is not recommended that flood control be provided in infiltration basins due to restrictions on allowable depth (to prevent compaction).

Storage Configuration/Depth

In an infiltration basin, surface storage is used to retain water for infiltration. In monitoring studies (Galli, 1990), one of the causal factors of failure was noted to be the depth of water retained in the basin. The weight of the water is thought to compact the basin decreasing its infiltration potential. The depth of storage should be limited to a maximum of 0.6 metres in order to minimize the compaction of the basin.

The length and width of the basin will be determined by the characteristics of the site in question (topography, size and shape). A desirable length-to-width ratio for an infiltration basin is 3:1 or greater. The appropriate minimum bottom area of the basin can be calculated based on Equation 4.3 with the porosity (n) set to 1.

Location/Setbacks

Groundwater mounding calculations may be required to ensure that infiltration basins do not interfere with sewage system leaching beds. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field. It is anticipated that calculations will be required in areas where the soils are marginally acceptable for infiltration. Given the high percolation rates ($\geq$ 60 mm/h) recommended for the implementation of infiltration basins, groundwater mounding generally should not be a problem.

The setbacks from wells specified in the Building Code for leaching bed systems should also be observed for infiltration basins.

Planting Strategy

The vegetation in an infiltration basin should be able to withstand periods of ponding and maintain or enhance the pore space in the underlying soils. There is much literature to suggest that deep rooted legumes increase porosity and enhance infiltration compared to other ground covers (e.g., rotation of oat and corn crops with alfalfa) (Bryant et al., 1986; Minnesota Pollution Control Agency, 1989). As such, the planting strategy should include grasses and deep rooted legumes.

Pre-treatment

Effective pre-treatment SWMPs include wet ponds, dry ponds, and wetlands. Unfortunately, these are of limited use for small (< 5 hectare) drainage areas. For small drainage areas, sand filters, bioretention areas, vegetated filter strips, grassed swales or oil/grit separators should be used to pre-treat stormwater which discharges to an infiltration basin, especially if road runoff is treated. Source controls should also be investigated (e.g., sanding/salting practices, public education with respect to street/driveway sediments) in areas where an infiltration basin is proposed.

Overflow/By-pass

A by-pass flow path/pipe should be incorporated into the design of an infiltration basin to convey high flows around the basin. This will necessitate the construction of a flow splitter upstream of the basin (see Section 4.7). The by-pass may also function as:

- the normal outlet until the site is stabilized (inlet to the basin is blocked off);
- the normal outlet during basin maintenance; and
- the normal outlet during winter/spring conditions.

Construction

Infiltration basins will only operate as designed if they are constructed properly. There are three main rules that must be followed during construction:

- Basins should be constructed at the end of the development construction;
- Smearing of the native material at the interface with the basin floor must be avoided and/or corrected by raking or roto-tilling; and
- Compaction of the basin during construction must be minimized.

Technical Effectiveness

The factors that contribute to failure of infiltration basins are:

- poor site selection (industrial/commercial land use, high water table depth, poor soil type);
- poor design (depth of ponding);
- poor construction techniques (smearing, over-compaction, basin operation during the construction period);
- large drainage area (high sediment loadings);
- lack of pre-treatment and/or by-pass; and
- lack of maintenance.

One of the main problems with centralized infiltration basins, like infiltration trenches, is that water from a large area is expected to infiltrate into a relatively small area. This does not reflect the natural hydrologic cycle and generally leads to problems (groundwater mounding, clogging, compaction).

As with other infiltration SWMPs, groundwater contamination and clogging of the basin are concerns. During the winter period, there will be an increased potential for clogging and groundwater degradation due to high sediment and salt loads. Pre-treatment and/or by-passing of winter flows may be required to address these concerns.

Infiltration basins should be designed with a by-pass system for large flows. Because of this requirement as well as the depth limitation to minimize soil compaction, infiltration basins are ineffective for water quantity control.

Infiltration basins may be incorporated in a stormwater management plan downstream from another end-of-pipe facility.

4.6.7 Filters

Filters are a relatively new type of end-of-pipe SWM facility for Ontario. They have been used extensively in parts of the United States with good success (Metropolitan Washington Council of Governments, 1992). Filters can be implemented above ground, or below ground as part of the storm sewer system infrastructure, and are generally intended for small drainage areas ($\leq$ 5 ha). Filters are a water quality SWMP and have no practical application for erosion or quantity control. In many applications, filters discharge to the storm sewer system. However, direct discharge to a watercourse is possible where there is sufficient topographic relief.

Types of Filters
Filters are versatile and come in many forms (Figures 4.33 to 4.37) including:

- surface sand filter;
- organic filter;
- underground sand filter;
- perimeter sand filter; and
- bioretention filter.

Surface and underground sand filters are the most common. Perimeter sand filters are especially useful around parking lots and because of their relatively large surface area, can be designed with lower head requirements. Organic filters can be designed as surface or subsurface devices. They employ a layer of peat in addition to the sand in order to enhance the removal of nutrients and trace metals. Filters using other media (such as iron filings) have also been used with some success. Bioretention filters are similar to conventional surface filters, but they allow the integration of open space and landscaping areas within the stormwater management facility.

Figure 4.33: Sand Filter Cross-section

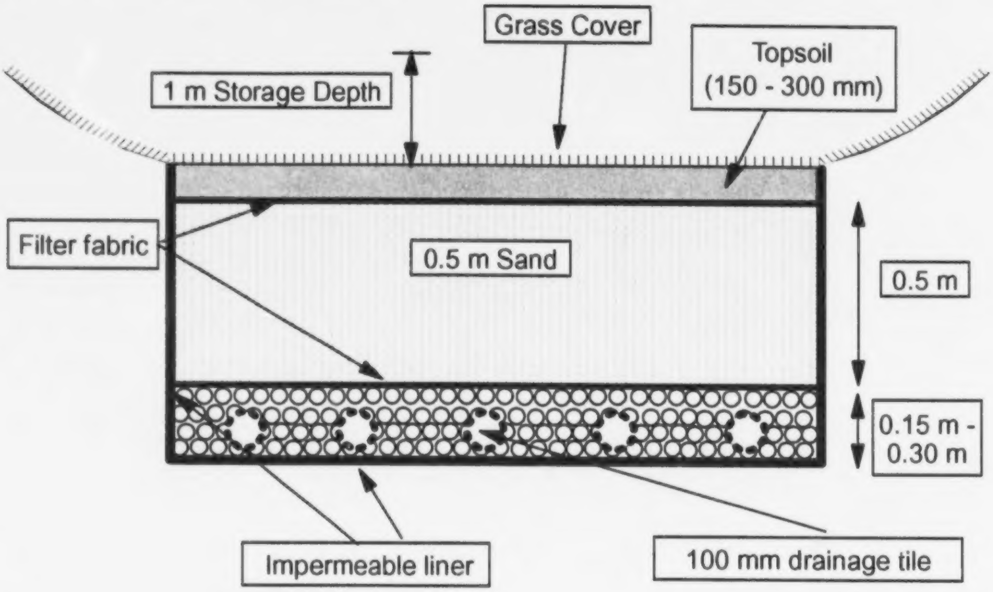

Grass Cover
1 m Storage Depth
Topsoil (150 - 300 mm)
Filter fabric
0.5 m Sand
0.5 m
0.15 m - 0.30 m
Impermeable liner
100 mm drainage tile

Figure 4.34: Peat Sand Filter Cross-section

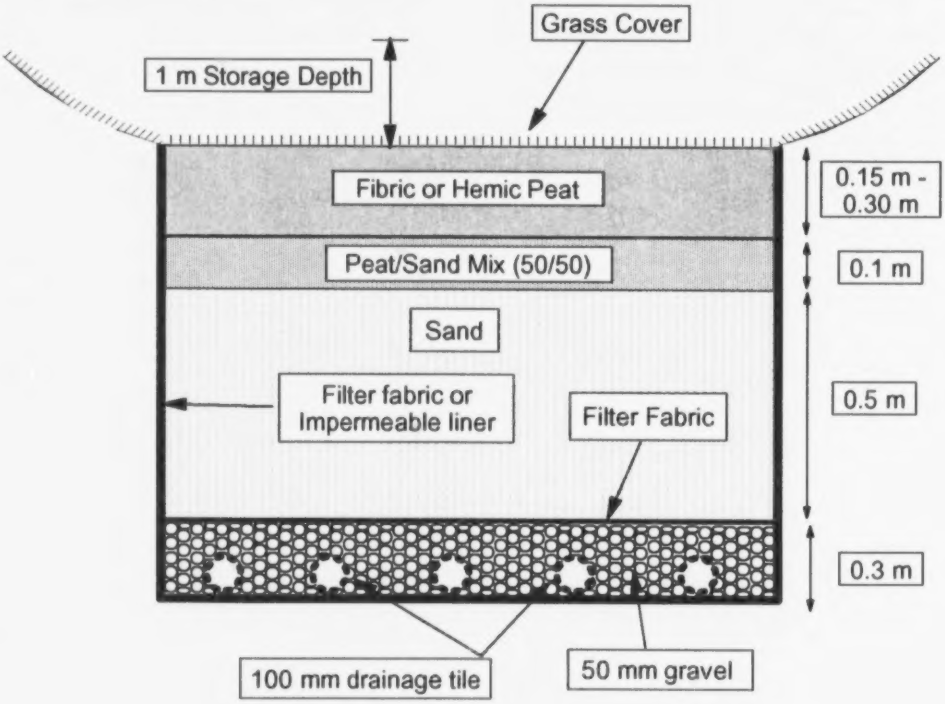
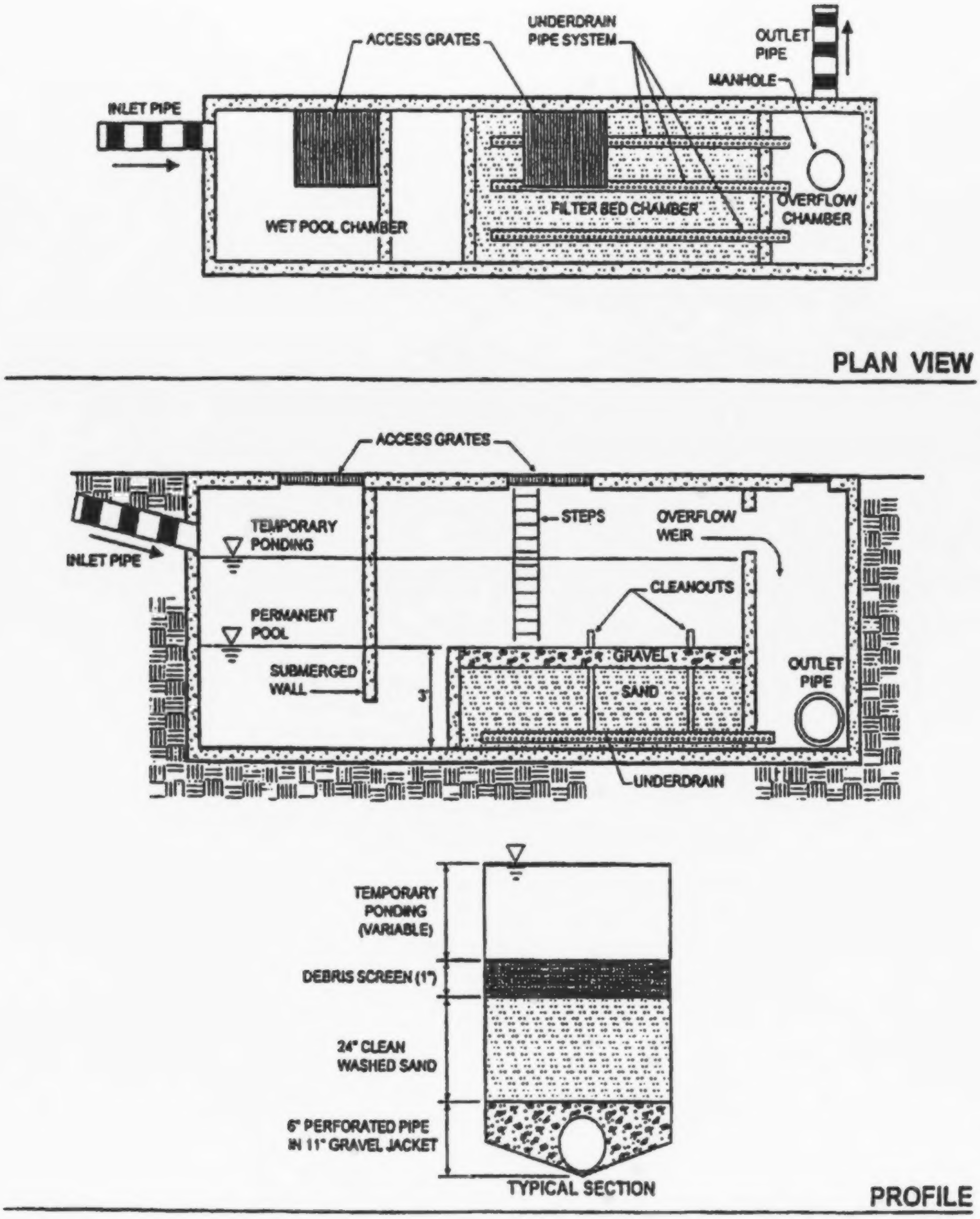
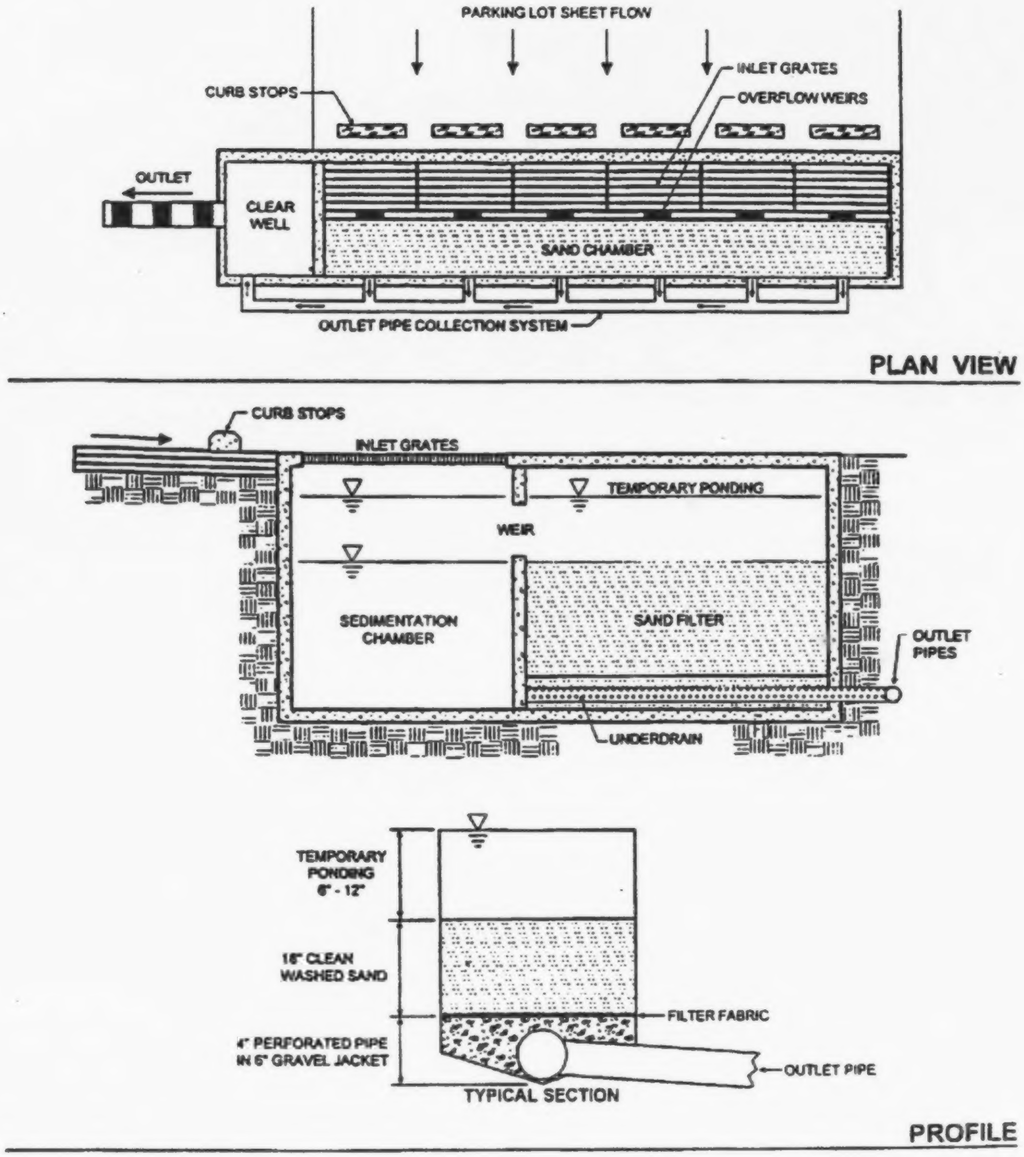

Grass Cover
1 m Storage Depth
Fibric or Hemic Peat
0.15 m - 0.30 m
Peat/Sand Mix (50/50)
0.1 m
Sand
Filter fabric or Impermeable liner
Filter Fabric
0.5 m
0.3 m
100 mm drainage tile
50 mm gravel

Figure 4.35: Underground Sand Filter


ACCESS GRATES
UNDERDRAIN PIPE SYSTEM
OUTLET PIPE
MANHOLE
INLET PIPE
WET POOL CHAMBER
FILTER BED CHAMBER
OVERFLOW CHAMBER
PLAN VIEW
ACCESS GRATES
STEPS
OVERFLOW WEIR
TEMPORARY PONDING
INLET PIPE
CLEANOUTS
PERMANENT POOL
GRAVEL
SUBMERGED WALL
3'
SAND
OUTLET PIPE
UNDERDRAIN
TEMPORARY PONDING (VARIABLE)
DEBRIS SCREEN (1")
24" CLEAN WASHED SAND
6" PERFORATED PIPE IN 11" GRAVEL JACKET
TYPICAL SECTION
PROFILE

Source: Maryland Stormwater Manual, Volume 1, 1998

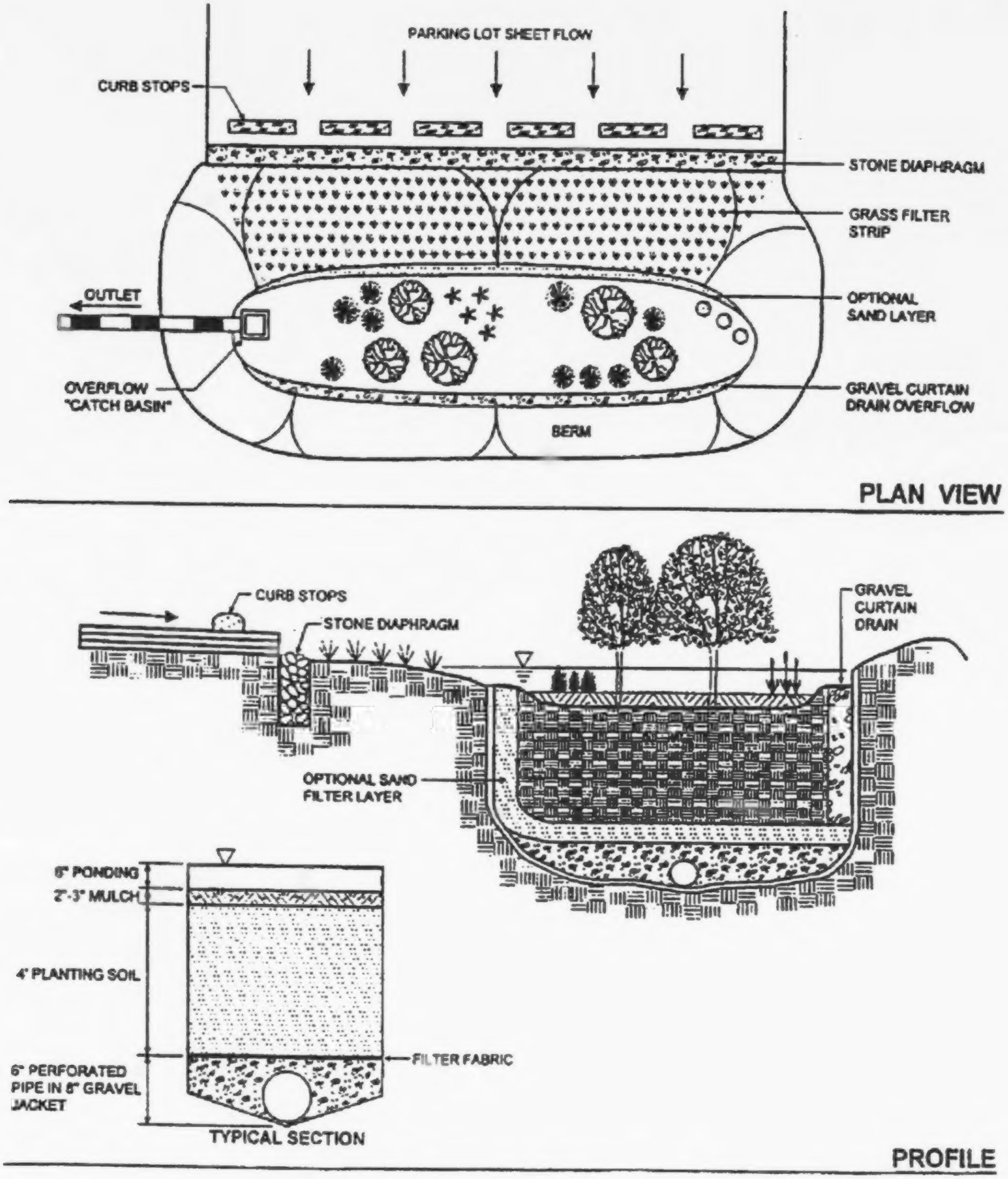
Figure 4.36: Schematic of a Perimeter Sand Filter


PARKING LOT SHEET FLOW
INLET GRATES
OVERFLOW WEIRS
CURB STOPS
OUTLET
CLEAR WELL
SAND CHAMBER
OUTLET PIPE COLLECTION SYSTEM
PLAN VIEW
CURB STOPS
INLET GRATES
TEMPORARY PONDING
WEIR
SEDIMENTATION CHAMBER
SAND FILTER
OUTLET PIPES
UNDERDRAIN
TEMPORARY PONDING 6" - 12"
18" CLEAN WASHED SAND
FILTER FABRIC
4" PERFORATED PIPE IN 6" GRAVEL JACKET
OUTLET PIPE
TYPICAL SECTION
PROFILE

The perimeter sand filter is most practical for small sites with flat terrain or a high water table.

Source: Maryland Stormwater Manual, Volume 1, 1998

Figure 4.37: Schematic of Bioretention Filter


PARKING LOT SHEET FLOW
CURB STOPS
STONE DIAPHRAGM
GRASS FILTER STRIP
OUTLET
OPTIONAL SAND LAYER
OVERFLOW "CATCH BASIN"
GRAVEL CURTAIN DRAIN OVERFLOW
BERM
PLAN VIEW
CURB STOPS
STONE DIAPHRAGM
GRAVEL CURTAIN DRAIN
OPTIONAL SAND FILTER LAYER
6" PONDING
2"-3" MULCH
4' PLANTING SOIL
FILTER FABRIC
6" PERFORATED PIPE IN 8" GRAVEL JACKET
TYPICAL SECTION
PROFILE

Bioretention combines open space with stormwater treatment.

Source: Maryland Stormwater Manual, Volume 1, 1998.

Filtration systems can be incorporated into most parking lot areas or commercial sites. The surface of the biofilter can be landscaped using trees, shrubs and riverstone or turf and integrated as an amenity within the overall landscape for the development. Biofilters can also be located beneath hard surface landscaped areas, such as courtyards, walkways and patios.

Design Guidance

There is limited operational experience with filters in Ontario. A summary of design guidance is provided in Table 4.10, based on experience in other jurisdictions. A more detailed discussion of this guidance is provided in the sections that follow.

Table 4.10: Filters – Summary of Design Guidance

Design Element	Design Objective	Minimum Criteria	Preferred Criteria
Drainage Area		< 5 hectares	
Treatment Volume	Provision of appropriate Level of protection (see Section 3.3)	As per Table 3.2 (infiltration)	
Pre-treatment	Longevity	Pre-treatment provided by sedimentation chamber or forebay, vegetated filter strip, swale or oil/grit separator	Sedimentation Chamber Volume: Winter By-pass: 25% of water quality treatment volume Winter Operation: 50% of water quality treatment volume
Storage Depth (head on filter)	Avoid filter compaction Protect vegetation	Subsurface sand and organic filters: 0.5 m Surface filters and bioretention: 0.15 m	

Table 4.10: Filters – Summary of Design Guidance (cont'd)

Design Element	Design Objective	Minimum Criteria	Preferred Criteria
Filter Media Depth	Filtering	Sand: 0.5 m Organic: 0.15 - 0.3 m peat 0.1 m peat/sand 0.5 m sand Bioretention: 1.0 - 1.2 m of planting soil	
Under-drain	Discharge	Minimum 100 mm perforated pipes bedded in 150 mm - 300 mm of 50 mm gravel	Winter operation: minimum 200 mm perforated pipes bedded in 300 mm of 50 mm gravel

Drainage Area

Filters should be implemented for drainage areas ≤ 5 ha.

Land Use

Filters may be employed for any land use, but because of their need for pre-treatment and ongoing maintenance, and the fact that they are not useful for erosion and quantity control, they are most often employed at commercial and industrial sites which have a high level of imperviousness. They can be designed to be part of the storm sewer system and thus can be combined with measures such as rooftop, parking lot or superpipe storage (for quantity control).

Volumetric Sizing

Water quality volumes to be used in the design are provided in Table 3.2 under the "infiltration" heading. Erosion and quantity control volumes are not applicable to this type of SWMP. The design should be such that at a minimum, the by-pass of flows should not occur below or at the peak runoff from a 4 hour 15 mm design event.

Storage Depth

It is recommended that the storage depth above a sand filter be limited to a maximum of 1 metre to reduce the potential for compaction of the sand layer. Storage depths for surface filters and bioretention areas should be limited to 0.15 m in order to protect the vegetation.

Media Depth

The recommended filter layer depth for most sand filters is 0.5 m. In bioretention filters, the planting soil layer is normally 1.0 to 1.2 m.

Filter Sizing

The area of the filter may be determined using the Darcy equation:

$$A = \frac{1{,}000\, V\, d}{k\,(h + d)\, t}$$ **Equation 4.12: Surface Area of Filter**

where A = surface area of the filter in m^2
V = design volume (m^3) derived from Table 3.2
d = depth (m) of the controlling filter medium (e.g., the sand or peat layer)
k = coefficient of permeability of the controlling filter media (mm/hr)
h = operating head of water on the filter (m)
t = design drawdown time in hours

Typical values for k are:

Sand:	45 mm/hr
Peat:	25 mm/hr
Leaf Compost:	110 mm/hr

Source: Maryland Stormwater Design Manual, Volume 1, 1998

Values of k for bioretention areas will depend upon the soil mix used and can be estimated from the particle size distribution. Drawdown time may range from 24 to 48 hours (24 hours is preferred).

Filter Lining

Filters are most commonly constructed with impermeable liners or within concrete structures to ensure that native material does not enter the filter and clog the pore spaces, and to prevent the filtered water from infiltrating into the native soil (i.e., prevent groundwater contamination).

Sand Filter Discharge

Water which percolates through the sand filter is collected by pervious pipes and conveyed to the outlet. The pervious pipes should be a 100 mm diameter drainage tile laid at the bottom of a 150 mm - 300 mm layer of 50 mm diameter gravel. The drainage tile should be installed with a maximum spacing of 1.2 m. A layer of non-woven geotextile filter fabric should be installed between the sand layer and the gravel layer to minimize the potential for fines to clog the pore spaces in the gravel. The perforated pipes can be wrapped in non-woven filter fabric (filter sock) to prevent sand from entering the perforated pipe.

Organic Filters

Organic filters most commonly include a layer of peat in the sand filter design. Peat has a high affinity for metals, nutrients and hydrocarbons, providing a greater level of water quality enhancement. The layer of peat is generally placed on top of the sand layer. The peat layer discharges to a peat-sand layer which provides a graduated increase in percolation to the sand layer. Fibric peat is recommended since it has a high percolation rate compared to more highly decomposed peats.

Pre-treatment

Pre-treatment is recommended for all filters. Ideally, a pre-treatment settling chamber should be incorporated into the design. The pre-treatment chamber should have a volume equal to 25% of the design water quality control volume. Other methods of pre-treatment include vegetated filter strips, grassed swales and oil/grit separators.

Overflow/By-pass

All sand filters should include an overflow or by-pass conveyance system. A by-pass system which routes flows around the sand filter is preferable to an overflow at the sand filter itself since there is no permanent pool in a sand filter to mitigate influent velocities. The influent pipe to the sand filter should be designed to accommodate the peak flow from a 4 hour Chicago distribution of a 15 mm storm.

Construction

Filters are very susceptible to clogging during the construction period. Therefore, construction practice should ensure that no runoff enters the filter until all contributing drainage areas have been stabilized (paved, vegetated, etc.).

Compaction can be a major problem in the construction of bioretention areas and may lead to failure of the facility. Where possible, excavation should be done with backhoes rather than loaders. Where loaders must be used, standard heavy construction equipment should be avoided in favour of light, wide-track or marsh-track equipment. Backfilling of the excavation with the planting mix should be done in 0.3 m or greater lifts. Light, wide-tracked or marsh-track equipment should be used for grading.

Winter Operation

Surface filters and bioretention areas are generally subject to the same problems as surface infiltration devices. Subsurface filters, while less susceptible than surface filters, also suffer in performance because of freezing in underdrain pipes or the filter medium. Filters which utilize organic media are particularly prone to freezing because they retain water.

Filters commonly receive runoff from parking areas and roads which are subject to sanding and salting. Pre-treatment is essential to avoid problems with clogging of the filter medium.

Most filter systems can be designed to operate seasonally using a by-pass. Where this is not done, it is recommended that the following measures be implemented in order to improve filter performance during cold weather:

- Increase underdrain pipes to a minimum of 200 mm diameter and slope at more than 1%;

- Flow controls and by-passes should be designed using weirs (rather than standpipes) where possible;

- Designs should include an oversized pre-treatment chamber (equivalent to 50% of the design treatment volume); and

- For surface filters or bioretention areas, runoff should be conveyed through a vegetated filter strip or swale (at least 5 m in length) prior to entering the filter area.

Maintenance
Removal of accumulated sediment from the pre-treatment chamber should be conducted regularly (at a minimum, when the accumulation depth exceeds 0.3 m). Silt/sediment should be removed from the surface of the filter when 2-3 cm has accumulated or when the drawdown time increases beyond 20% of the design value. The upper layer of the filter material (e.g., 0.1 to 0.15 m) should be removed and replaced with clear material when accumulated sediment is removed from the filter.

Technical Effectiveness
Filters are a relatively new type of end-of-pipe SWM facility for Ontario. They have been used extensively in parts of the United States with good success (Metropolitan Washington Council of Governments, 1992).

Filters are effective in removing pollutants, resistant to clogging (if pre-treatment is provided) and are generally easier and less expensive to construct/retrofit than infiltration trenches. Pre-treatment and longevity are critical factors that may vary from design to design. It is therefore recommended that they be used for pre-treatment, post-treatment, and in terms of water quality control that they be implemented as part of a multi-component approach. Filters are a water quality SWMP and have no practical application for erosion or flood control.

4.6.8 Oil/Grit Separators

Oil/grit separators (OGS) are used to trap and retain oil and/or sediment in detention chambers, usually located below ground. They operate based on the principles of gravity-based sedimentation for the grit, and phase separation for the oil. There is minimal attenuation of flow in oil/grit separators since they are not designed with extended detention storage. Like filters, they have no infiltration capability.

Separators are often used as spill controls, pre-treatment devices or end-of-pipe controls as part of a multi-component approach for water quality control. They are typically used for small sites but sizing and design are dependent on the function they are to fulfil. Whether separators are to be used as pre-treatment devices or in the overall design as a part of a multi-component system, it is essential to select appropriate equipment based on their demonstrated capabilities. There have been numerous applications of OGS since the 1980s and there are a variety of both proprietary and non-proprietary oil/grit separators on the market ranging from chambered designs to manhole-types.

Design Guidance
Many suppliers provide design guidance specific to their products. This guidance is normally application-dependent. The designer should be fully aware of the assumptions and methodology used in formulating performance predictions as they may have implications for the design. The application and design should be discussed with approval agencies.

Oil/grit separators are typically used for small drainage areas (< 2 hectares). As a spill or grit control device, they are particularly well-suited to conditions where space is constrained. Since the majority of spills are small in volume and are not weather dependent, separators provide an effective means of control. Oil/grit separators are beneficial for industrial and commercial sites, and large parking areas or transit facilities where there is a higher risk of spills and a greater opportunity for sediment build-up. OGS are also often used to provide an inspection access as part of a storm sewer discharge control program (e.g., sewer use by-law). Typical applications include:

- automobile service station parking areas;
- selected areas at airports;
- some industrial or commercial areas;
- pre-treatment for other SWMPs;
- infill/retrofit developments;
- areas susceptible to spills of material lighter than water (bus depots, transfer stations); and
- inspection structures for private industrial or commercial sites that drain to the municipal sewer system.

Many end-of-pipe stormwater practices benefit from pre-treatment, typically in the form of pre-settling in pond forebays or settling chambers, or biofiltration in swales, vegetative filter strips or sand filters. Pre-treatment can also be provided by oil/grit separators. Separators are particularly well suited for use with subsurface SWMPs without pre-treatment chambers incorporated into their design. The benefits of pre-treatment include the extension of the operational life of stormwater management facilities adversely impacted by sediment, the extension of maintenance intervals and the prevention of oil sheen.

For stormwater quality control, oil/grit separators may be applied as one element of a multi-component approach unless it is determined that it can achieve the desired water quality as a stand-alone device on a site-specific basis. In a multi-component approach there is a series of stormwater quality measures for water quality improvement. A multi-component approach can often be expected to provide a higher level of improvement.

Potential applications of oil/grit separators may be for water quality control for re-development projects in an urban core or stormwater quality retrofits for an existing development. However, in new residential developments where space is not as constrained, the selection of SWM practices may be governed by having to meet erosion and flood control objectives as well as water quality objectives.

Oil/grit separators generally form part of the underground storm sewer infrastructure. Their use is typically not as constrained by space considerations, bedrock or groundwater levels, or soil conditions (although they have foundation bedding requirements similar to manholes and underground tanks).

Winter Operation

There is relatively limited data on the effects of cold weather and winter runoff on the performance of oil/grit separators. Depending on the depth and location of the installation, separators may be susceptible to freezing, which will reduce or eliminate their effectiveness (e.g., by causing more frequent by-pass or overflow). Designs that retain water between events may also be susceptible to salt stratification resulting in short-circuiting and reduced detention times and removal rates. Further studies are needed to verify the impacts of salt stratification on different OGS designs. In the interim, more frequent maintenance (e.g., removal of retained water) may be employed during the winter months to enhance performance.

Technical Effectiveness

There have been many refinements to existing designs of oil/grit separators and new designs have come to the market. Research and monitoring of many of the devices has been conducted by manufacturers of the devices and government agencies. The reported results have been quite variable (both excellent and very poor) and appear to be highly dependent on study design, specific site conditions, sizing of the device relative to the contributing drainage area, particle size distribution and the varying flow conditions under which the tests were conducted. The higher removal efficiencies generally corresponded to the presence of high percentages of sand or heavy sediments in the stormwater or lower flows into the separator. Therefore, if reported results are to be used as a reference, comparable factors should be thoroughly considered.

It is noteworthy that a key factor in assessing the performance of OGS is the level at which by-pass occurs. A by-pass is generally provided for OGS as discussed in section 4.7. Under conditions of high stormwater flow, the overall solids removal efficiency of the separator usually decreases since the stormwater which is by-passed receives no treatment. Oil/grit separators will be required to be sized to capture and treat at least 90% of the runoff volume that occurs for a site on a long-term average basis for water quality objectives of 'enhanced protection.' For water quality objectives of 'normal protection' and 'basic protection,' at least 85% of the total runoff volume that occurs for a site will be required to be captured and treated. In each application, the facilities are still required to meet the water quality objectives of long-term average removal of 80%, 70% and 60% of suspended solids in the total runoff volume for 'enhanced,' 'normal' and 'basic' protection levels, respectively. Suspended solids removal efficiency is to be calculated based on 100% of the total runoff volume for all the storm events that occur for the site on a long-term average basis. For example, for a site that requires a normal level of protection (70% suspended solids removal) with a facility that only captures 85% of the runoff volume, the facility would have to achieve a performance of 82% suspended solids removal for that 85% of the volume. The average efficiency would be 70% = (85% of the volume × 82% efficiency) + (15% of the volume × 0% efficiency).

The multi-component approach has been confused in the past with the term "treatment train." In the multi-component approach, a series of stormwater quality practices are used to meet water quality objectives. The treatment train approach is premised on providing control at the lot level and in conveyance followed by end-of-pipe controls. A treatment train is required to meet the multiple objectives of water balance, water quality, and erosion and flood control in an overall stormwater management strategy. Lot level and conveyance controls can reduce end-of-pipe storage requirements for erosion control and are the best means of achieving water balance objectives. Water quality improvement and quantity control for small storms are secondary benefits. End-of-pipe controls are required to meet water quality, and erosion and flood control objectives in most circumstances.

4.7 Flow Splitters/By-Passes

While many stormwater facilities are designed to meet multiple objectives (e.g., water quality, erosion control, quantity control), some SWMP designs are intended for water quality control only. In such cases, the design capacity of the SWMP will normally be less than the capacity of the stormwater conveyance system. It is often necessary to by-pass larger flows in order to prevent problems with the SWMP (e.g., re-suspension of sediment) or damage to the facility (e.g., compaction of soil, etc.). Flow splitters are used to direct the runoff from a water quality storm into an end-of-pipe SWM facility, but by-pass excess flows from larger events around the facility into another SWMP or directly into the receiving waters. As a general rule, flow splitters are generally provided for:

- extended detention dry ponds (without a forebay);
- constructed wetlands (without a forebay);
- infiltration trenches/basins;
- oil/grit separators; and
- filters.

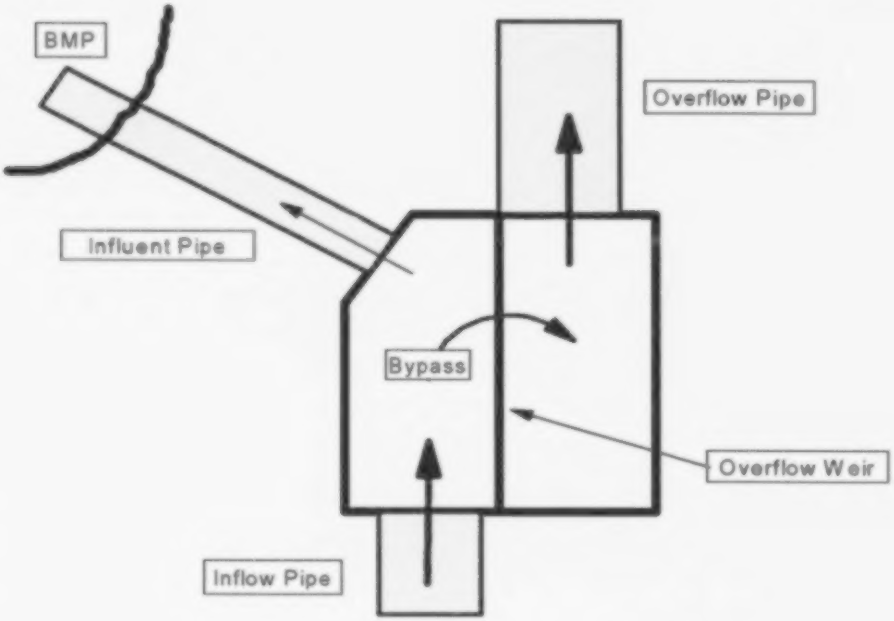
Most Ontario municipalities will not accept mechanical/electrical controls on stormwater management facilities due to the potential for operational and maintenance problems associated with numerous real-time control systems within a municipality's jurisdiction. Therefore, the preferred flow splitter design operates on hydraulic principles.

The design of a hydraulically operated flow splitter must account for backwater conditions in the SWM facility, the hydraulic potential into the facility at the design by-pass rate, and the potential for flow reversal during the recession limb of a storm.

Design Guidance

The typical flow splitter design is shown in Figure 4.38. Water is conveyed to the SWM facility via a first flush pipe. Once the facility reaches its design capacity, water backs up in the influent pipe and, hence, the flow splitter itself. As the water level reaches the by-pass elevation, stormwater begins to by-pass the SWM facility. The by-pass is generally accommodated by a weir in the flow splitter structure.

Figure 4.38: Storm Sewer Flow Splitter



The by-pass elevation, by-pass capacity and influent pipe capacity dictate how the flow splitter will operate.

By-pass elevation

There are two main methods for setting the by-pass elevation. The first is to set the by-pass elevation such that the design storm is captured at the maximum by-pass rate. Using this method, the design water level in the end-of-pipe SWM facility would be equal to the depth of water in the by-pass structure necessary to achieve the maximum anticipated by-pass rate. Although this ensures that the maximum design water level in the SWM facility is never exceeded, it also causes the by-pass to operate for storms smaller than the design event.

The second method is to set the by-pass elevation equal to the design storage elevation in the end-of-pipe SWM facility. Using this method, the facility will only start to by-pass once it has captured the design runoff volume. Although this ensures that the design volume will be captured before by-pass occurs (given the influent pipe capacity is not exceeded), it also means that the water level in the SWM facility will exceed the design level for large infrequent storms which utilize the by-pass. This method is preferred since it is conservative. An example of the operation of this type of flow splitter is provided in Figure 4.39.

By-pass Capacity

Given that the by-pass elevation is set equal to the design storage elevation in the end-of-pipe SWM facility, the maximum elevation in the facility depends on the rate of by-pass with depth. For example, it would be ideal (from a hydraulic perspective) to have a long by-pass weir, such that a small depth increment resulted in a large flow (and very little increase in depth in the facility). However, large weirs result in large flow splitter structures which are expensive. Therefore, an optimum balance between depth increase and by-pass design should be sought recognizing that the increase in storage depth due to a flow splitter is not prolonged.

In end-of-pipe SWM facilities with only an extended detention outlet, the flow splitter has the potential to cause flow reversal. Flow reversal is the flow of water out of a SWM facility via the flow splitter structure. Flow reversal has the potential to occur when the flow over the by-pass subsides quickly while the flow out of the pond is governed by a slow extended detention release. In these instances, water levels in the facility above the by-pass elevation can force water back into the flow splitter and over the by-pass structure during the recession limb of a by-pass storm.

In order to minimize flow reversal, several guidelines can be followed:

- provide an overflow outlet above the design water level in the SWM facility;
- minimize the depth of by-pass to achieve the desired by-pass rate; and
- maximize the head losses between the facility and the by-pass to minimize the hydraulic potential into the SWM facility.

It should be recognized, however, that the provision of an overflow outlet will result in more water being conveyed into the pond, which defeats the original purpose of the flow splitter. Therefore, the provision of an overflow outlet is not recommended as a standard practice to minimize flow reversal.

The operation of the by-pass must be assessed for the design event used to size the upstream pipe network. This event will vary from municipality to municipality (2 year, 5 year, 10 year storm). In some cases, the upstream pipe network will be oversized due to overland flow constraints and may convey the 100 year storm. In all cases, the splitter operation during the pipe design event must be assessed.

Figure 4.39: Operation of a Hydraulic Flow Splitter

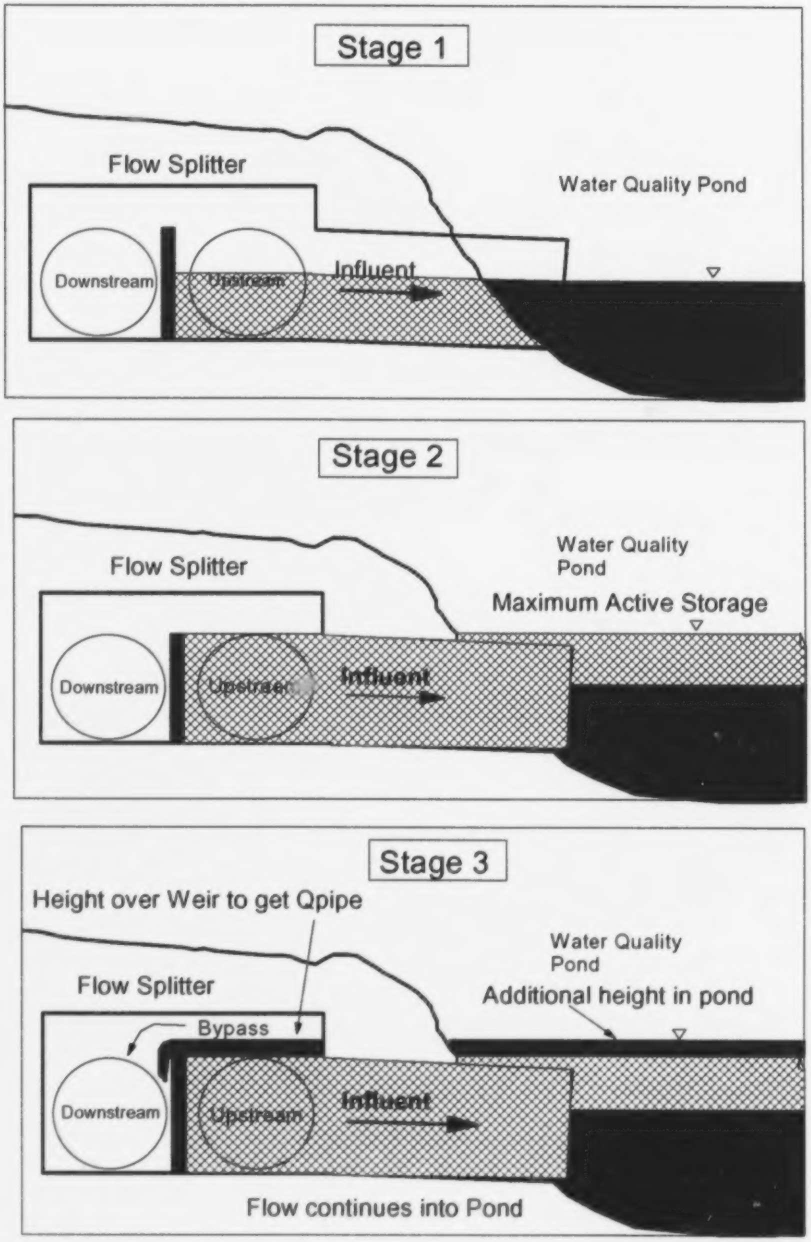
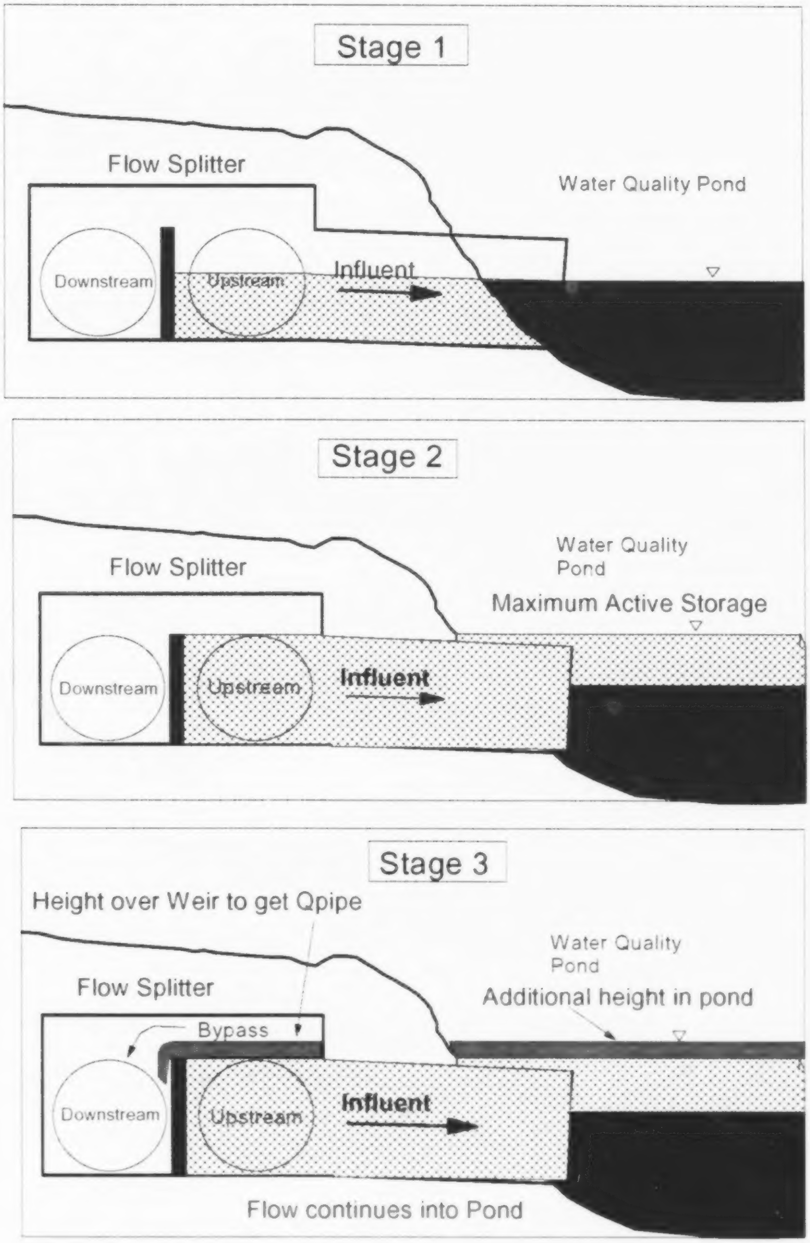

Stage 1
Flow Splitter
Water Quality Pond
Downstream
Upstream
Influent
Stage 2
Water Quality Pond
Flow Splitter
Maximum Active Storage
Downstream
Upstream
Influent
Stage 3
Height over Weir to get Qpipe
Water Quality Pond
Flow Splitter
Additional height in pond
Bypass
Downstream
Upstream
Influent
Flow continues into Pond

Figure 4.39: Operation of a Hydraulic Flow Splitter


Stage 1
Flow Splitter
Water Quality Pond
Downstream
Upstream
Influent
Stage 2
Water Quality Pond
Flow Splitter
Maximum Active Storage
Downstream
Upstream
Influent
Stage 3
Height over Weir to get Qpipe
Water Quality Pond
Flow Splitter
Additional height in pond
Bypass
Downstream
Upstream
Influent
Flow continues into Pond

Influent Pipe Capacity
The capacity of the influent pipe into the SWM facility determines when a by-pass occurs for intense storms, and it adds to the head loss between the by-pass and the pond (this determines the hydraulic potential into the pond once the by-pass begins to operate).

The design of the influent pipe depends on the intensity of the water quality storm in question. Event simulations of a 2 hour Chicago distribution of a 25 mm storm produce peak flows which are similar to a 2 year storm (**Note:** A 25 mm storm can be a 100 year event if it occurs in a very short time-frame.) Since the design inflow is supposed to be representative of a frequent event, the 2 hour distribution is overly conservative. It is recommended that a 4 hour Chicago distribution be used to determine a peak flow for the sizing of the first flush pipe.

In pond systems, this will equate to the peak flow from a 25 mm storm. A 15 mm storm should be used for end-of-pipe infiltration systems and filters, and a 10 mm storm should be utilized for vegetated filter strips.

Maintenance By-pass
In cases where the facility accommodates high flows as well as low flows, a flow splitter is not required. A maintenance by-pass is recommended, however, to facilitate maintenance operations in the urban SWMP if conventional "in-the-dry" maintenance techniques are to be used. Such a by-pass is not needed if the intention is to use hydraulic dredging or other "wet" techniques in maintenance.

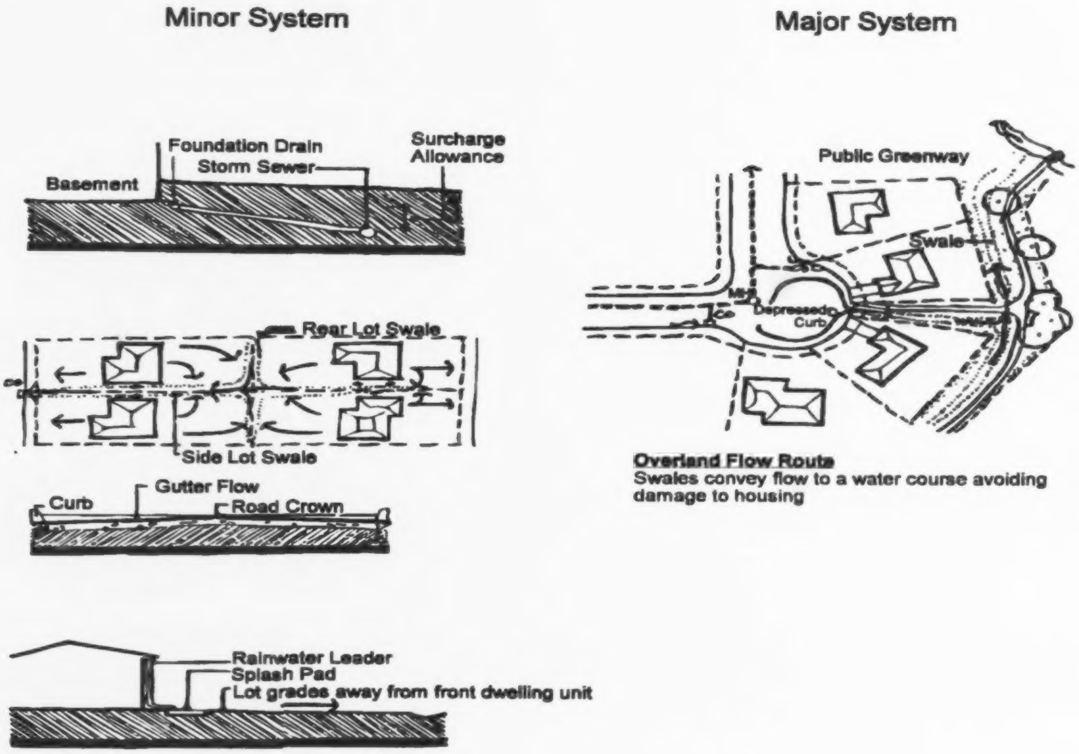
The by-pass can be provided by a second pipe in a manhole upstream of the SWMP itself. The second pipe would be closed/blocked during the normal operation of the pond. During the maintenance periods, the inlet to the pond would be closed and the maintenance pipe opened. The maintenance pipe could either discharge to an overland flow path around the SWMP, or to the outlet via an underground pipe system. If water is conveyed to the SWMP by swale, a separate maintenance swale must be constructed around the SWMP from the pond inlet, and the design must incorporate movable gates to re-direct flows into the maintenance swale during maintenance periods.

4.8 Major/Minor System Design

Stormwater management includes recognition and design of the major and minor systems (Figure 4.40). The minor system conveys frequent runoff events while the major system conveys infrequent runoff events. The major and minor systems exist for pre- and post-development conditions, whether designed or not.

Generally, the division between the major and minor systems in urban areas is defined by municipal or township standards. Standards define the location within road allowances, the materials of construction, the physical dimensions of the common drainage elements (i.e., curb

Figure 4.40: Major/Minor Systems


Minor System
Major System
Foundation Drain
Storm Sewer
Surcharge Allowance
Basement
Rear Lot Swale
Side Lot Swale
Curb
Gutter Flow
Road Crown
Rainwater Leader
Splash Pad
Lot grades away from front dwelling unit
Public Greenway
Swale
Depressed Curb
Overland Flow Route
Swales convey flow to a water course avoiding damage to housing

Source: Urban Drainage Design, Ontario Ministry of Natural Resources, Ministry of the Environment et al, 1987

heights, catchbasins, road slopes, lot grading, etc.), and the procedures to be employed in sizing storm sewer or ditches. Not all urban areas have been designed employing a major/minor system approach. In some municipalities, only the minor system has been designed while the major system has evolved.

For pre-development conditions, the minor system is defined as the stream or low flow portion of the watercourse. The floodplain can be viewed as the major system. The division between major and minor systems is a function of the hydrologic processes that occur within the watershed. Many factors including topography, vegetation, geology, groundwater and meteorology define the low flow and floodplain capacities of a watercourse.

Stormwater management systems reflect the physical characteristics of the development that it supports whether the development land use is residential, commercial, industrial or transportation. Well-designed drainage systems require a set of criteria that will mitigate potential natural, economic and social environment impacts.

Good design ensures the following:

- That upstream runoff is conveyed through the development with minimal impacts to upstream people, properties and natural environments;
- That site runoff is conveyed to a secure outlet with minimal impacts to the development site users and property; and
- That site runoff has minimal impacts on downstream people, properties and natural environments.

For the development site, drainage design objectives include protecting property from surficial flooding, protecting basements from surcharge or backwater flooding, and street drainage is designed for convenience and conveyance. Criteria or standards are generally defined by the municipalities and townships that assume drainage, sewage and road works for new development.

4.8.1 Minor System

For urban areas, the minor system is composed of lot grades, ditches, backyard swales, roof leaders, foundation drains, gutters, catchbasins, and storm sewers. Traditionally, minor systems have been designed to reduce the inconvenience to the public from runoff events.

Sewer Sizing

Each municipality will set the standards for the sizing of storm sewers. Generally, storm sewers are sized to convey runoff from storms with return periods ranging from 2 years to the 10 years. The storm sewers are sized to convey frequent events and prevent nuisance flooding. Most municipal standards will set a minimum diameter to prevent siltation and clogging. Minimum storm sewer sizes are approximately 250 mm diameter.

The Rational Method (Equation 4.8) is the most commonly used method to determine peak flow rates to be conveyed by the storm sewer system. The parameters utilized in the Rational Method are usually defined by the municipality or township. Generally, the capacity of storm sewers is determined using Manning's equation.

Roof Downspouts

Roof downspouts should discharge to lawns to encourage infiltration.

Catchbasin Inflows

The total storm sewer inflows from catchbasins and roof leaders should be determined to prevent surcharging of the storm sewer system and possible basement flooding. Controls can be placed in catchbasins to reduce inflows to the full flow capacity of the storm sewer system.

Catchbasins do not catch 100% of the road flow. As the road flow increases, the top width of flow becomes greater than the width of the catchbasin grate. Water will either by-pass or overshoot the catchbasin grate. As a result, runoff will continue down the road to the next catchbasin. For low flows, catchbasins will capture approximately 80 to 100% of the gutter flow. For high flows, approximately 70% of the road flow will by-pass the grate.

Catchbasin Spacing

Catchbasins are usually placed at low points within the road system. The catchbasins are spaced to prevent gutter/road flow from by-passing the catchbasin grate.

Maximum Sewer Capacity

The storm sewer reaches maximum capacity when the critical depth of flow is equal to the storm sewer diameter. Increasing the storm sewer slope beyond the critical slope will not increase the storm sewer capacity. The critical slope will be a function of the peak flow rate, the roughness of the storm sewer and the storm sewer diameter.

Maximum and Minimum Flow Velocities

Flow velocities should be limited to approximately 6 m/s, while minimum flow velocities should exceed 1 m/s. Slower flow velocities will tend to deposit silt and sediment.

Basement Flooding

Each municipality will have criteria to prevent basement flooding for new urban development. The following are generic criteria:

a. Minimum lot grades, i.e., 2%;
b. Minimum 1% swale slopes at lot boundaries;
c. Basement floor elevations above the 100 year hydraulic gradeline in the storm sewer system;
d. Basement floor elevations above the backwater level in the minor system produced by the Regulatory Flood in the major system;
e. Basement floor elevations above high groundwater levels;
f. Building invert opening elevations above the maximum site ground elevation; and
g. Foundation drain collectors not susceptible to backwater or surcharging, etc.

4.8.2 Major System

For urban areas, the major system includes natural streams, valleys, swales, artificial channels, roadways, stream road crossings and ponds. The major system conveys runoff from infrequent events that exceed the minor system capacity. The major system will exist whether it has been designed or not. For good design, the major system will reduce the risk to life and property damage by providing overland flow routes to a safe outlet. Most flow routes will follow the natural topography.

Although the primary of function of roads is to convey vehicular traffic, roads can be used to convey runoff. Standards for using the roadway as a floodway are set by the local municipality. The following are general criteria:

a. On arterial roads, the depth at the crown shall not exceed 0.15 m;

b. On local roads, flow should not overtop the curbs;

c. On collector roads, 1 lane should be left free from flooding;

d. On arterial roads, 1 lane should be left free from flooding in each direction;

e. Flow should not cross roads except for minor storms;

f. Low points along the road grade should not exist unless the low points are conveying flow to the major system;

g. The product of flood depth at the gutter multiplied by the flow velocity shall be less than 0.65 m^2/s; and

h. At regular intervals along the road, major storm runoff should be conveyed to a watercourse or a major channel.

4.9 Modelling Techniques

Lot level and conveyance controls result in reductions in the quantity of runoff which must be treated by end-of-pipe controls. Stormwater modelling (provided in support of development applications) can reflect these reductions. Event modelling for sizing end-of-pipe flooding controls should also recognize and incorporate end-of-pipe storage provided for quality and erosion control. Table 4.11 summarizes modelling approaches which may be used, and further detail is provided in the sections which follow. Guidance is also provided in terms of reductions in the volumetric requirements provided in Table 3.2. It should be noted that any change in storage requirements with respect to Table 3.2 represents a reduction in the volume of active storage required (not permanent pool).

Table 4.11: Urban Development Stormwater Management Practice Integration*

SWMP	Water Quality Benefits		Flooding Benefits	Erosion Benefits
	Event Modelling	**Table 3.2**	**Event Modelling**	**Event Modelling**
Lot Grading	increase pervious depression storage for lot areas by 0.5 mm for every 0.5% reduction in lot grade from 2%	water quality active storage reduced by 5 m^3/ha for every 0.5% reduction in lot grade from 2%	increase pervious depression storage for lot areas by 0.5 mm for every 0.5% reduction in lot grade from 2%	increase pervious depression storage for lot areas by 0.5 mm for every 0.5% reduction in lot grade from 2%
Roof Leader Ponding	model roof areas separately, increase impervious depression storage by (ponding volume/roof area)	water quality active storage reduced by $\sum$ ponding volume (m^3) which cannot exceed that required to store runoff to the ponding area	model roof areas separately, increase impervious depression storage for roof areas by (ponding volume/roof area)	model roof areas separately, increase impervious depression storage for roof areas by (ponding volume/roof area)

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* *The storage provided by stormwater lot level and conveyance controls may be reduced by a longevity factor(see text for further detail) when assessing end-of-pipe SWM facility storage requirements for water quality, quantity and erosion.*

Table 4.11: Urban Development Stormwater Management Practice Integration (cont'd)

SWMP	Water Quality Benefits		Flooding Benefits	Erosion Benefits
	Event Modelling	**Table 3.2**	**Event Modelling**	**Event Modelling**
Roof Leader Soakaway Pits	model roof areas separately increase impervious depression storage by (volume of voids in pit/roof area) OR model pits as reservoirs (route flow to pits using reservoir routine)	water quality active storage reduced by volume of voids in soakaway pit which cannot exceed that required to store runoff to the pit	model roof areas separately increase impervious depression storage by (volume of voids in pit/roof area) OR model pits as reservoirs	model roof areas separately increase impervious depression storage by (volume of voids in pit/roof area) OR model pits as reservoirs
Pervious Pipes	route flow through a DIVERT HYD or similar routing routine to divert exfiltrating flows based on orifice calculations. Exfiltrated flows should be routed through a reservoir based on feasible trench sizing with overflows added back into the pipe	water quality active storage reduced by exfiltration media storage	route flow through a DIVERT HYD or similar routing model to divert exfiltrating flows based on orifice calculations. Exfiltrated flows should be routed through a reservoir with overflows added back into the pipe	route flow through a DIVERT HYD or similar routing model to divert exfiltrating flows based on orifice calculations. Exfiltrated flows should be routed through a reservoir with overflows added back into the pipe
Pervious Catchbasins	route water quality event using a reservoir routine based on feasible catchbasin size	water quality active storage reduced by pervious catchbasin storage	route water quantity events using a reservoir routine based on feasible catchbasin size and evaluate incidental benefits	route erosion event using a reservoir routine based on feasible catchbasin size and evaluate incidental benefits

Table 4.11: Urban Development Stormwater Management Practice Integration (cont'd)

SWMP	Water Quality Benefits		Flooding Benefits	Erosion Benefits
	Event Modelling	Table 3.2	Event Modelling	Event Modelling
Infiltration Trench	route water quality event using a reservoir routine based on feasible trench sizing	water quality active storage reduced by infiltration trench storage	route water quantity event using a reservoir routine and evaluate incidental benefits	route erosion event using a reservoir routine and evaluate incidental benefits
Enhanced Grass Swales	route water quality event using a reservoir routine	water quality active storage reduced by enhanced grass swale storage	route water quantity events using a reservoir routine based on enhanced grass swale design	route erosion event using a reservoir routine based on enhanced grass swale design
Vegetated Filter Strip	route water quality event using a reservoir routine with a weir outlet to size storage based on the allowable weir flow depth	N/A	size vegetated filter strip based on water quality event and evaluate incidental benefits during water quantity storm	size vegetated filter strip based on water quality event and evaluate incidental benefits during the erosion storm
Wet Pond	route water quality event using a reservoir routine and size storage to obtain desired detention time	water quality active storage reduced by wet pond active storage	route quantity events with erosion/quality reservoir routine and size quantity storage	route erosion event using water quality reservoir routine and size erosion storage
Dry Pond	route water quality event using a reservoir routine and size storage to obtain desired detention time	water quality active storage reduced by dry pond storage	size dry pond based on quality and evaluate incidental benefit during quantity storms	route erosion event using water quality reservoir routine and size erosion storage

Table 4.11: Urban Development Stormwater Management Practice Integration (cont'd)

SWMP	Water Quality Benefits		Flooding Benefits	Erosion Benefits
	Event Modelling	**Table 3.2**	**Event Modelling**	**Event Modelling**
Dry Pond with forebay	route water quality event using a reservoir routine and size storage to obtain desired detention time	water quality active storage reduced by dry pond storage	route quantity events with erosion/quality reservoir routine and size quantity storage	route erosion event using water quality reservoir routine and size erosion storage
Wetland	route water quality event using a reservoir routine and size storage to obtain desired detention time	water quality active storage reduced by wetland storage	size wetland based on quality and evaluate incidental benefit during quantity storms	route erosion event using water quality reservoir routine and size erosion storage
Wetland with Forebay	route quality event using a reservoir routine and size storage to obtain desired detention time	water quality active storage reduced by wetland storage	route quantity events with erosion/quality reservoir routine and size quantity storage	route erosion event with quality reservoir routine and size erosion storage
Sand Filter	route water quality event using a reservoir routine and size storage to prevent overflows	water quality active storage reduced by sand filter storage	size filter based on quality and evaluate incidental benefits during quantity storms (route quantity event through filter)	route erosion event using reservoir routine based on filter size and evaluate incidental benefits
Infiltration Basin	route water quality event using a reservoir routine based on feasible basin sizing	water quality active storage reduced by infiltration basin storage	size basin based on feasibility and evaluate incidental benefits from basin during quantity storms	route erosion event using water quality reservoir routine and size erosion storage

4.9.1 Reduced Lot Grading

Recommended lot grading changes include the provision for a typical standard slope (2 - 5%) within 2 - 4 metres of any buildings and a flatter slope (< 2%) for the remaining lot area. These grading changes are only appropriate if the site is naturally flat.

Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
Event modelling using an altered pervious depression storage will account for the effect of the flatter grading on the required end-of-pipe water quality, quantity and erosion storage.
The water management benefit derived from these grading changes given a typical lot (12 m frontage by 30 m depth) is estimated to be an additional 0.5 mm of pervious depression storage for every 0.5% reduction in the typical 2% grading standard (i.e., 1.5% grade = +0.5 mm, 1.0% grade = +1 mm, 0.5% grade = +1.5 mm).

The depression storage can be further adjusted based on a "longevity factor" which takes into account public acceptance and landowner alterations to individual site grading (Equation 4.13). It is recommended that a longevity factor of 0.75 be used for developments that implement reduced lot grading.

$$DSP = 4.67 + (2 - G)f$$

Equation 4.13: Adjusted Pervious Depression Storage

where DSP = pervious depression storage (mm)
G = 0.5% (lot grading)
f = 0.75 (longevity factor)

A higher or lower factor could be requested by a regulatory agency (municipality, MNR, MOE, CA) based on the agency's experience with people altering site grading patterns. There is a tendency for people to make minor alterations.

Water Quality Storage Requirements Based on Table 3.2
The effects of reduced lot grading on the water quality storage requirements presented in Table 3.2 can be estimated by Equation 4.14.

$$V = (A - LL) \times S + LL \times [S - ((2 - G) \times 10 \times f)]$$

Equation 4.14: Lot Grading Adjustment

where V = volume of water quality storage required (m^3)
A = total development area (ha)
LL = development area with flat lot grading (ha)
S = original water quality requirement from Table 3.2 (m^3/ha)
G = lot grading (%)
f = longevity factor

If the second term on the right hand side of Equation 4.14 is negative (i.e., more storage is provided by the reduced lot grading than is required), it should be set to zero. The development area with reduced lot grading (LL) should not include the road right-of-ways.

4.9.2 Roof Leader Discharge to Surface Ponding Areas

Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
The benefits from discharging roof leaders to on-site surface ponding areas can be estimated by altering the impervious depression storage for rooftop areas.

The change in impervious depression storage depends on the volume of surface ponding provided and the roof area being controlled. Equation 4.15 provides an estimation of the adjusted impervious depression storage. The use of Equation 4.15 implies that the roof areas will be modelled separately since the impervious depression storage of driveways and roads should not be altered.

$$\mathrm{DSI} = \frac{\mathrm{SPV}}{\mathrm{RA}} \times 1{,}000 \times f$$

Equation 4.15: Impervious Depression Storage

where DSI = adjusted impervious depression storage (mm)
SPV = volume of surface ponding storage (m^3)
RA = rooftop area (m^2)
f = longevity factor

The longevity factor in Equation 4.15 accounts for the public acceptance of local ponding areas on their property. It is recommended that a longevity factor of 0.75 be used for surface ponding storage for rooftop leader discharges.

Water Quality Storage Requirements Based on Table 3.2
The effects of surface storage for roof leader discharges on the water quality storage requirements presented in Table 3.2 can be estimated by Equation 4.16.

$$V = [(A - RS) \times S] + [(RS \times S) - (SPV \times f)]$$

Equation 4.16: Roof Leader Surface Storage Adjustment

where V = volume of water quality storage required (m^3)
A = total development area (ha)
RS = rooftop area + surface ponding area (m^2)
S = water quality requirement from Table 3.2 (m^3/ha)
SPV = volume of surface ponding storage (m^3)
f = longevity factor

If the second term on the right hand side of Equation 4.16 is negative (i.e., more surface storage is provided for runoff to the ponding area than is required), it should be set to zero. The rooftop area as defined in Equation 4.16 includes the surface storage area (and surface drainage tributary to the ponding area).

4.9.3 Roof Leader Discharge to Soakaway Pits

Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
The water management benefit from the implementation of soakaway pits can be determined by adjusting the impervious depression storage using Equation 4.15 and the method described in Section 4.9.2. The term SPV (surface ponding volume) would be replaced with the volume of void space in the soakaway pit (e.g., 40% of the gravel storage volume with 50 mm gravel).

Another method to calculate the benefit of soakaway pits with respect to water quality, erosion and quantity control is to model them as reservoirs. The areas with soakaway pits would be lumped together and modelled separately. The flow from this area would be routed through a reservoir. The outflow rate from the reservoir would depend on the configuration of the pit and the volume of water it contains. A rating curve can be calculated based on the storage and flow rate as shown in Equation 4.17. It is recommended that the longevity factor be based on the percolation rate of the surrounding soils due to the lack of widespread experience with the implementation of soakaway pits. Table 4.12 provides recommended longevity factors based on native soil percolation rates.

$$Q = f \times \left(\frac{P}{3{,}600{,}000}\right) \times (2LD + 2WD + LW) \times n$$

Equation 4.17: Soakaway Pit Rating Curve

$$V = LWD \times n \times f$$

(Rating Curve Storage Volume)

where Q = flowrate (m^3/s) for a given storage volume (V)
f = longevity factor
P = native soil percolation rate (mm/h)
L = length of the soakaway pit (m)
W = width of the soakaway pit (m)
D = depth of water in the soakaway pit (m)
V = volume of water in the soakaway pit (m^3)
n = void space in the soakaway pit storage layer

Table 4.12: Longevity Factors for Conveyance Media

Soil Percolation Rate (mm/h)	Longevity Factor (f)
< 25	0.5
> 25 but < 100	0.75
> 100	1

Water Quality Storage Requirements Based on Table 3.2
The effect of soakaway pits for roof leader discharge on the water quality storage requirements presented in Table 3.2 can be estimated using Equation 4.16. The term SPV (surface ponding volume) would be replaced with the volume of void space in the soakaway pit (e.g., 40% of the gravel storage volume with 50 mm gravel).

4.9.4 Pervious Pipes

Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
The benefits provided by the implementation of pervious pipes with respect to water quality, erosion and quantity storage are more difficult to quantify since the exfiltration is dependent on many factors (pipe slope, number of perforations, size of perforations, depth of flow). Paul Wisner and Associates (1994) indicated that the exfiltration can be reasonably modelled using the orifice equation with a variable orifice coefficient. The coefficient varies from 0 to a maximum of 0.63 depending on the perforation size and depth of flow. However, this calculation must be performed along the pipe as water flows through the system since water is continuously exfiltrating.

The results from the Wisner study were used to derive a steady state equation for the rate of exfiltration with flow through the pipe. The exfiltration rate for any given flow in a pipe can be approximated using Equation 4.18. This equation indicates that the exfiltration rate depends on the inflow rate, the size and number of perforations along the pipe and the slope of the pipe. The results for Equation 4.18 were compared to the laboratory tests performed in the above-noted study. The simplified relationship is less accurate than continuous simulation. However, it provides a reasonable approximation of the pipe flow – exfiltration relationship.

$$Q_{exft} = (15A - 0.06S + 0.33)\,Q_{inf}$$

Equation 4.18: Exfiltration Discharge

where Q_{exft} = exfiltration flow through pipe perforations (m^3/s)
A = area of perforations/m length of pipe (m^2/m)
S = slope of perforated pipe (%)
Q_{inf} = flow through the perforated pipe (m^3/s) (longitudinally)

Equation 4.18 was developed based on the measured flows in a 300 mm diameter pipe with 12.7 mm and 7.9 mm perforations. The use of Equation 4.18 for larger diameter pipes, or pipes with much larger perforations, should be scrutinized.

Based on equation 4.18, a rating curve can be estimated for the exfiltration flow as a function of flow in the perforated pipe system. Using this relationship, a hydrograph diversion routine (such as DIVERT HYD in OTTHYMO) can be used to determine the split in flows that are conveyed by the perforated pipe system.

The diverted flows that represent exfiltration should be further routed through a reservoir routine based on the exfiltration storage, and native material. Equation 4.17 can be used to determine the routing reservoir rating curve for the exfiltrated water. The term "2 W D" in Equation 4.17 should be set to zero in this application since the exfiltration storage is a linear system. The overflows from the reservoir signify that the exfiltration storage is full, and should be added back to the pipe flow. This can be accomplished using a second DIVERT HYD command.

Water Quality Storage Requirements Based on Table 3.2
If Table 3.2 is used, the volume of exfiltration storage can be compared directly with the infiltration storage required to achieve the desired level of water quality protection for the appropriate level of imperviousness. The required infiltration storage should be adjusted based on the longevity factor (Table 4.12).

4.9.5 Pervious Catchbasins

Water Quality, Erosion, and Flood Control Storage Requirements Based on Modelling
The benefit with respect to water quality, quantity and erosion control resulting from the installation of pervious catchbasins can be estimated using the same methodology as that derived for soakaway pits (i.e., reservoir routing using Equation 4.17). The modelling of pervious catchbasins is easier than soakaway pits since the lot and road areas do not have to be separated into different basins.

Water Quality Storage Requirements Based on Table 3.2
The effects of pervious catchbasins on the water quality storage requirements presented in Table 3.2 can be estimated by Equation 4.19.

$$V = (A \times S) - (CBV \times f)$$

Equation 4.19: Pervious Catchbasin Adjustment

where V = volume of water quality storage required (m^3)
A = development area draining to pervious catchbasins (ha)
S = water quality requirement from Table 3.2 (m^3/ha)
CBV = volume of pervious catchbasin storage (m^3)
f = longevity factor

The longevity factor should be estimated from Table 4.12.

4.9.6 Sand Filter

The sizing of a sand filter for water quality control is based on the runoff to be treated (either from Table 3.2 or the runoff from a 15 mm storm) and the filtration area.

Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
The outflow from the sand filter should be based on Equation 4.20.

$$Q = f \times \left(\frac{P}{3{,}600{,}000}\right) \times (LW \times n)$$

Equation 4.20: Sand Filter Discharge

where Q = flowrate (m^3/s) out of the sand filter
f = longevity factor
P = percolation rate for sand (mm/h)
L = length of the filter (m)
W = width of the filter (m)
n = void space in the sand filter (typically 0.25)

Only the length and width (bottom area) are used in Equation 4.20 since the assumption is made that the filter is contained within an impermeable liner causing all of the discharge to be collected in the perforated pipes at the bottom of the filter.

The longevity factor in Equation 4.20 should be based on the percolation rate of the sand used from Table 4.12.

In order to assess the incidental benefits of sand filters in reducing erosion and flooding storage requirements, the sand filters should be modelled using a reservoir routine. An assumption of constant outflow with increasing storage can be made as a conservative assumption. Some marginal increase in flow will have to be assumed with increasing storage recognizing the mathematical instabilities inherent in most reservoir routines. The rating curve above the water quality storage volume should reflect conveyance without attenuation to simulate the by-passing of the filter.

Water Quality Storage Requirements Based on Table 3.2
The volume of storage provided by the filter should be adjusted using the longevity factor (Table 4.12) based on the filter media and may be compared directly with the water quality storage requirements in Table 3.2.

4.9.7 Infiltration Trenches

The sizing of an infiltration trench is based on the runoff to be treated (either from Table 3.2 or the runoff from a 15 mm storm) recognizing that a smaller size may be implemented based on the physical constraints imposed by the site.

Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
The same methodology used for sand filters to account for incidental flooding and erosion benefits can be used for infiltration trenches. The assumption of using the bottom area of the trench is still reasonable since the infiltration trench has a sand filter layer for pollutant removal (i.e., the trench should be designed to convey water through the bottom).

The percolation rate (P) and longevity factor (f) in Equation 4.20 should be assessed for the native material below the infiltration trench recognizing that the surrounding native material will be the limiting conveyance media. The porosity in Equation 4.20 should be equal to the void space in the storage media (0.4 for clear stone). The rating curve above the water quality storage volume should reflect conveyance without attenuation to simulate the by-passing of the trench.

Water Quality Storage Requirements Based on Table 3.2
The volume of infiltration storage provided by trench should be adjusted using the longevity factor (Table 4.12) based on the native material and may be compared directly with the storage required in Table 3.2.

4.9.8 Enhanced Grass Swales

Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
Enhanced grass swales have a permanent check dam to hold back water during small events. The check dam acts as a weir during larger events and can be modelled as a reservoir. The rating curve for the reservoir can be determined based on the stage storage relationship upstream of the check dam in the swale and the weir equation (Equation 4.4). Given the small storage volume contained in one swale, and the likelihood for numerous swale areas, the storage volume from several swales should be lumped together in this assessment (i.e., downstream check dam controls lumped storage).

In order to ensure that numerical instability does not occur in the routing routine, there should be a positive discharge from the swale as the storage increases (even below the elevation of the check dam). The discharge from the swale below the elevation of the check dam can be calculated using Equation 4.20 where the term LW represents the contact area between the water and the swale (i.e., the wetted perimeter of the swale below the check dam). The percolation rate (P) in Equation 4.20 should be assessed for the native soil material and the porosity should be set to 1.

The longevity factor for enhanced grass swales should be 1.0 since they are not directly dependent on infiltration for operational performance.

Water Quality Storage Requirements Based on Table 3.2
The effects of enhanced grass swales on the water quality storage requirements presented in Table 3.2 can be estimated by Equation 4.19. The term CBV would be replaced with the storage provided upstream of the enhanced grass swale at the elevation of the check dam.

4.9.9 Vegetated Filter Strip

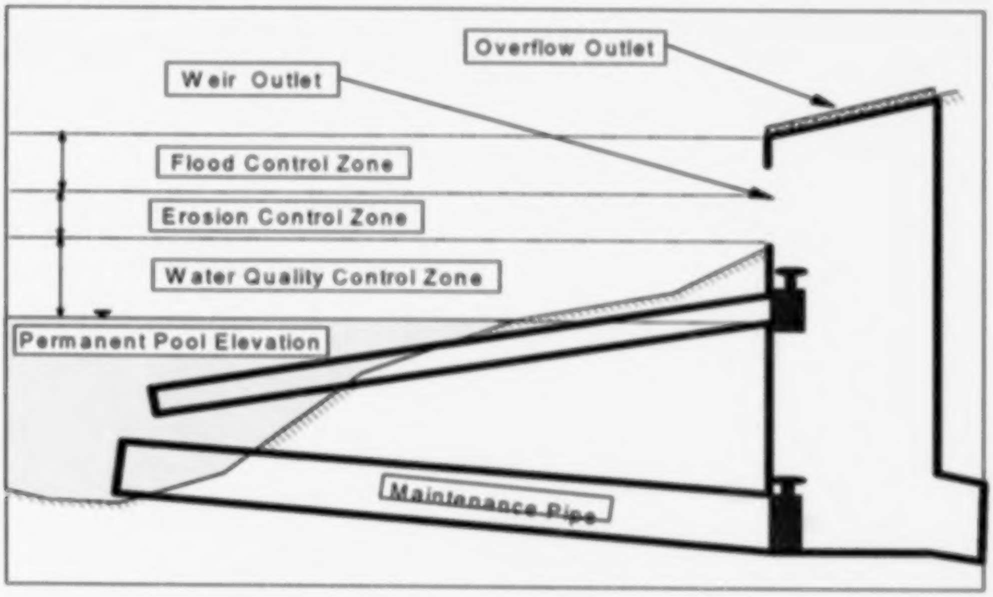
Water Quality, Erosion and Flood Control Storage Requirements Based on Modelling
The sizing of the level spreader is based on the conveyance of the peak flow rate from a 10 mm (4 hour Chicago storm) with a depth of less than 100 mm based on the weir equation (Equation 4.4). If a short spreader is utilized there may be a need for storage upstream of the level spreader to ensure that flow depths through the vegetated filter strip do not exceed 100 mm. A reservoir routine can be used to assess the water quality storage required upstream of the level spreader.

If there is appreciable storage designed upstream of the level spreader, the incidental flood and erosion control benefits from the vegetated filter strip can be assessed using the reservoir routine (the vegetated filter strip should not be designed specifically for erosion and flood control). In most cases, however, it is expected that the level spreader will be designed such that there is minimal upstream storage required. It should be assumed that there are no flood and erosion control benefits resulting from the implementation of a vegetated filter strip under these conditions.

4.9.10 Wet Pond

The benefits with respect to erosion and flooding from the implementation of water quality storage in a wet pond are easily calculated. The required water quality storage (either from Table 3.2 or the Subwatershed Plan) can be modelled as a storage reservoir in an event model. The values in Table 3.2 are based on a 24 hour detention. Additional points can be added to the reservoir to simulate erosion control (if different from water quality control) and flood control. This methodology can be used if quality, quantity and erosion are accommodated in the one pond. Multiple reservoir routines can be implemented for separate facilities. Figure 4.41 illustrates the zones in a combined quality and quantity wet pond. It is noteworthy that in many instances the water quality control zone may be eliminated since its function is served by the erosion control zone (e.g., each have similar drawdown times – typically 24 hours).

Figure 4.41: Multiple Objective Pond Outlet



4.9.11 Dry Pond

A dry pond should not be used for combined quality and quantity control unless a forebay is included in the design due to the potential for re-suspension and scour of previously settled pollutants. The benefits with respect to erosion and flooding from the implementation of water quality storage in a dry pond which incorporates a forebay are calculated in the same manner as for a wet pond.

4.9.12 Wetland

A wetland should not be used for combined quality and quantity control unless a forebay is included in the design due to the potential for re-suspension and scour of previously settled pollutants. The benefits with respect to erosion and flooding from the implementation of water quality storage in a wetland which incorporates a forebay are calculated in the same manner as for a wet pond.

4.9.13 Infiltration Basin

The assessment of incidental flooding and erosion control benefits derived from the implementation of an infiltration basin is identical to that for an infiltration trench. The bottom area of the basin in Equation 4.20 represents the contact area of water and the basin floor. As such, a reservoir routing rating curve can be developed based on the grading in the basin (i.e., stage-storage-contact area relationship) and Equation 4.20 (with the porosity set to 1). The rating curve above the water quality storage volume should reflect conveyance without attenuation to simulate the by-passing of the basin.

4.10 New Environmental Technologies

As the "state-of-the-art" of stormwater management evolves there will be new technologies not discussed in this manual. For the evaluation and assessment of new environmental technologies, the Ministry has a program called the New Environmental Technology Evaluation (NETE) program. Under the program, the ministry will assess/evaluate a new technology by reviewing the information and data submitted by an applicant, conducting relevant literature searches on similar technologies and utilizing relevant engineering and technical knowledge/expertise of reviewing engineers and/or scientists. Following the assessment, a NETE "Opinion Letter of Technology Assessment" or a "Certificate of Technology Assessment" will be issued to the applicant commenting on the technical merits of the technology, its potential to meet jurisdictional environmental standards and potential areas of application. There is a fee for the evaluation.

A guide for applicants for the NETE program is available on the web-site:

- http://www.ene.gov.on.ca/envision/gp/3552.pdf.

Questions or requests for further information may be directed to:

The Manager
Technology Standards Section
Standards Development Branch
Ministry of the Environment
7th Floor, 40 St. Clair Avenue West
Toronto, Ontario   M4V 1P5

The NETE evaluation is not considered an approval or implied approval of the technology and it in no way removes or limits the obligation to obtain the necessary environmental approvals under the *Ontario Water Resources Act* or the *Environmental Protection Act* for an application of the technology. The ministry approval process ensures the applicability of the technology against site-specific performance and environmental requirements.

5.0 INFILL DEVELOPMENT

5.1 General

Infill projects can range in size from a single lot to the complete redevelopment of significantly larger areas. Many forms of infill development can be more intensive than previous uses and have higher levels of imperviousness (e.g., more pavement), runoff rates, and contaminant loading per unit of area. In many cases, areas surrounding the new infill development were built before the need for stormwater controls was recognized and are already experiencing stormwater management problems. Although the development of single, individual infill sites may not have significant impacts, the development of many individual sites can have cumulative effects and exacerbate or create problems at the subwatershed and watershed level including flooding, erosion, or water quality degradation.

The focus of this chapter is on infill areas less than 5 hectares where storm sewer infrastructure exists.

Applying stormwater management practices in developed areas can be a challenge. Land availability and cost often limit stormwater management options in infill situations. Stormwater controls in infill situations are frequently implemented on private property and owners are responsible for their maintenance. Municipalities can generally require owners to maintain these controls; however, the proliferation of numerous, small, scattered facilities may be undesirable from a management and operations perspective.

The proponent is responsible for meeting all approval requirements from relevant agencies.

Some infill developments may meet the conditions of the Approval Exemption Regulation (AER). Under the AER 525 passed in September 1998, sub-sections 53 (1) and (3) of the *Ontario Water Resources Act* do not apply. Therefore, MOE approval is not required for the establishment, alteration, extension or replacement of or a change in a stormwater management facility that:

- is designed to service one lot or parcel of land;
- discharges into a storm sewer that is not a combined sewer;
- does not service industrial land or a structure located on industrial land; and
- is not located on industrial land.

Infill Development Plan/Subwatershed Rehabilitation Plan
The preparation of an Infill Development Plan or Subwatershed Rehabilitation Plan (i.e., a Subwatershed Plan for a developed area) is the preferred approach in addressing stormwater water quality and quantity concerns associated with infill development issues particularly in municipalities where significant growth is expected from infill development (See Appendix F).

These types of plans take into consideration local factors such as:

- physical conditions (e.g., practical infiltration levels);
- infrastructure capacity;
- anticipated growth due to infill or intensification; and
- the opportunities for retrofitting or rehabilitating stormwater management systems.

Appropriate planning leads to a more efficient and coordinated use of resources in implementing stormwater controls and greater watershed benefits. Typically these types of plans are lead by local municipalities and public input is an important component in establishing priorities and objectives.

5.2 Infill Development SWM Approaches

Infilling is most common in the following land use categories:

- commercial (normally processed under a site plan approval);
- industrial (normally processed under a site plan approval); and
- small residential (usually less than 6 lots – processed under a plan of subdivision application).

The cumulative effect of infill development may result in higher peak stormwater flows, increased erosion, and greater contaminant loading. Peak flow control and water quality can potentially be addressed at the site level for commercial/industrial infill. Because erosion control requires that a larger volume of runoff must be stored for an extended period (approximately 24 hours), it is more difficult in infill situations.

Although the scale of the anticipated infill development may not be sufficient to warrant the development of an Infill Development Plan or Subwatershed Rehabilitation Plan, municipalities have typically considered the following options to manage stormwater from residential and commercial/industrial infill development: no control, minimum runoff capture, conveyance/end-of-pipe controls, and off-site systems.

5.2.1 Residential Infill

In general, SWMPs to small scale residential infill are limited to lot level controls because of the small area of land in individual ownership and the presence of existing stormwater conveyance infrastructure. In virtually all cases, having residential roof leaders discharge to ponding areas is an applicable practice (e.g., lawn). Where soils permit, soakaway pits or infiltration trenches can be used, although problems with long-term maintenance and longevity may occur because of the private ownership and potential for lack of maintenance. Reduced lot grading can be used where the soils permit, but the acceptability of this type of control should be confirmed by the local municipality (some municipal standards require a minimum 2% slope).

i) No Control
This approach is not accepted by some municipalities without at least an assessment of infiltration potential and is limited to small residential infill developments (in some cases only a single lot).

ii) Minimum Runoff Capture
Requires the proponent to capture all runoff from a small design rainfall event (typically 5 mm) and retain it on site until it infiltrates or evaporates. If feasible, lot level source controls should be used for all residential infill to mitigate cumulative erosion impacts. Where soils and municipal by-laws permit, roof drainage to soakaway pits, infiltration trenches or cisterns, and flatter lot grading may be used. Roof leader discharge to pervious areas should be applied even to single lots unless physically infeasible.

iii) Conveyance/End-of-Pipe Controls
In addition to the lot level controls, some small residential infill projects may provide the opportunity to apply conveyance controls. In situations where new stormwater infrastructure is required and soil conditions are favourable, swale drainage or pervious pipe systems may be considered for clean stormwater. The decision to implement these types of controls should be confirmed by the municipality. Generally, end-of-pipe controls are not applicable to residential infill and are rarely used.

iv) Off-Site Systems (OSS) to address cumulative stormwater impacts
On-site stormwater management is generally preferred. However, in certain situations it may be ineffective or impractical because of physical constraints. In these cases, off-site systems (OSS) may be considered for all residential infill beyond a single lot. Off-site treatment can help address water quality, erosion and flood control impacts caused by development within a watershed. Proponents are still responsible to ensure that they meet all legislative requirements including the federal *Fisheries Act*.

OSSs can be used in combination with minimum runoff capture and conveyance/end-of-pipe controls. A number of municipalities have used the approach of requesting a financial contribution toward the development of SWM at another location elsewhere in the watershed and have used various formulas to calculate required financial contribution (see Section 5.3).

5.2.2 Commercial/Industrial Infill

The opportunities to apply SWMPs to small-scale commercial/industrial infill are usually greater than those found in residential infill. However, land availability and costs as well as municipal zoning requirements (e.g., number of parking spaces, etc.) can be limiting factors. Surface SWM facilities, such as wet ponds, constructed wetlands and infiltration basins, often are not viable because of the relatively large amount of surface area required. Rooftop, parking lot and superpipe storage, while generally applicable, are not accepted by some approval agencies. Lot level controls should be used to the extent possible to supplement end-of-pipe controls. Guidance for the design and sizing of each of these types of SWMPs is provided in Chapter 4. The majority of other SWMs can be applied depending on stormwater quality, soil conditions, and the individual development's design. Table 5.1 lists the types of SWMPs that can be used in infill situations, type of control they provide, and conditions which limit their use.

i) No/Minimal Controls

This approach is normally only considered for small industrial/commercial infills comprising less than 0.3 hectares (**Note:** this cut-off may be modified to reflect specific municipal conditions and policies); and may be coupled with off-site systems (see iv below). Roof leader discharge to pervious areas should be applied if physically feasible and practical (unless there is potential for contamination from roof top). Oil/grit separators may be used for areas that have a higher potential for spills (such as gas stations).

ii) Minimum Runoff Capture

This approach requires the proponent to capture all runoff (small design rainfall event – typically 5 mm) and retain it on site (runoff volume is usually either infiltrated or evaporated). This approach may be used for clean water where soils permit and infills are greater than 0.3 hectares.

In highly impervious commercial and industrial infill developments, the potential usefulness of this approach is dependent on the ability to infiltrate the runoff where there are no concerns about groundwater contamination (i.e., stormwater must be clean).

iii) Conveyance/End-of-Pipe Controls

Certain conveyance and end-of-pipe controls are commonly used in commercial/industrial infill (Table 5.1). In most cases, quantity controls (e.g., rooftop or parking lot storage) are required for commercial/industrial infills because of sewer system capacity and flooding concerns. The use of rooftop and parking lot storage is not accepted by some approval agencies in terms of the overall flood storage requirements for a subwatershed; however, storage is often useful for municipalities due to limited sewer capacity.

End-of-pipe controls for peak flow control should be mandatory where there is concern for downstream storm sewer capacity or where there are flooding concerns and no opportunity for centralized flood control facilities. Facilities for erosion control should only be applied where there is a clear need or where there is a potential to combine the requirements for water quality/quantity and erosion control (e.g., a dry pond). Even where there is a plan for use of off-site systems (OSS) within the subwatershed, additional water quality controls may be required where there is a high potential for wash-off of contaminants (e.g., oil and grease at gas stations, etc.).

Table 5.1: SWMPs Applicable to Infill Development

SWMP Type	*Type of Control*	**Comments**
Rooftop Storage	Peak Flow	Application dependent upon building design
Parking Lot Storage	Peak Flow	Application dependent upon site grading
Superpipe Storage	Peak Flow	Application dependent upon invert of street storm sewer
Dry Pond (quantity control)	Peak Flow	Application dependent upon available surface area
Pervious Pipe	Water Quality	Application dependent upon soils. May be combined with superpipe to provide both peak flow and some water quality control
Swales[*]	Water Quality	Most useful where infiltration capacities are high
Pocket Wetland[*]	Water Quality	Requires high water table to sustain wetland
Dry Pond (24 hr. retention)	Erosion	Application dependent upon available surface area. Minimum orifice size may govern feasibility.
Dry Pond (48 hr. retention)	Water Quality and Erosion	Application dependent upon available surface area. Minimum orifice size may govern feasibility.
Infiltration Trench[*]	Water Quality	Application dependent upon soil infiltration capacity and protection of groundwater
Sand or Organic[*] Filters	Water Quality	Generally applicable
Bioretention Filters[*]	Water Quality	Generally applicable
Oil/Grit Separators[*]	Spills/Water Quality[*]	Generally applicable

[*] *Should be used as part of a multi-component approach including more than one SWMP when used as a water quality control unless it is demonstrated on a case-by-case basis that the water quality criteria can be met.*

iv) Off-Site Systems (OSS) to address stormwater cumulative impacts

Off-site systems (OSS) have been used where on-site stormwater management practices are ineffective or impractical because of physical constraints. In order to try and offset stormwater impacts from the development, the project proponent may be required to make a financial contribution to a SWM system at another location within the same subwatershed. A number of municipalities have used this approach using various formulas to calculate the required financial contribution. Although on-site controls are typically preferred, an OSS can be used as an alternative to help address water quality, erosion and flood control impacts caused by development within a watershed. Proponents are still responsible to ensure that they meet all legislative requirements including the federal *Fisheries Act*.

An OSS may be considered for all commercial/industrial infills greater than 0.3 hectares if a plan for subwatershed rehabilitation is in place or a set of priority projects has been established for the subwatershed.

5.3 Off-Site Systems (OSS) and Financial Contribution

Off-site system (OSS) SWMPs are most effective within the context of an Infill Development or Subwatershed Rehabilitation Plan so that any funds collected in lieu of on-site facilities can be applied to suitable projects within the same subwatershed. If a plan has not been developed, issues of concern, priorities and suitable SWMPs to address these concerns need to be identified within the watershed. Funds should be targeted at projects identified in the Rehabilitation Plan or a priority list that addresses impacts to which infill contributes (e.g., water quality, erosion, and/or flood controls).

An off-site system (OSS) program involving financial contributions can be developed by using the *Development Charges Act, 1997*. Storm drainage is an eligible and fully fundable service under section 5(5).3 of the *Development Charges Act, 1997* (DCA). However, any determination of an appropriate "financial contribution" via the DCA for stormwater costs or any other DCA-eligible purpose must meet all the rules and requirements of that Act for determining charges.

In general, development charges may only recover the "net growth related capital costs" of eligible facilities. These eligible costs must be determined according to various rules and limitations set out in the Act covering such aspects as: the definition of "capital cost" for the DCA purposes; the "average service standards" provision (which limits chargeable costs); and various rules dealing with the allocation of costs between existing and new (re)developments where both may benefit from the works in question.

Any such provisions for development charges for stormwater facilities (related to infill development or otherwise) must be contained in a DCA by-law in effect in any given municipality and justified with supporting background study before individual charges can be imposed.

By-law provisions (the original by-laws and amendments adding any new charges) are subject to several process requirements such as public meetings and, if there are objections, to an Ontario Municipal Board (OMB) appeals process.

5.3.1 Potential Approach in Using and Financing Off-Site SWMPs

The following steps may be helpful in determining the use of Off-Site System (OSS) stormwater management practices for infill development and their funding through financial contributions. Modifications may be appropriate to meet specific conditions or policies within a municipality or region.

1) Develop Watershed Rehabilitation Plan/Priority Projects List to identify and prioritize projects in subwatershed so that an OSS approach provides a strategic benefit greater than that expected from the scattered use of site controls. Otherwise, funds collected could be spent without benefit to the subwatershed. The projects identified should be located in the same subwatershed as the source of funding and the scale of the project should reflect the expected pace of infill. The projects which make up the plan should include all types (water quality, and erosion and flood control) for which a financial contribution is to be collected.

2) Adopt a financial contribution formula which reflects the local conditions and the anticipated types of projects. A simple formula based on site area and imperviousness has been used (see section 5.4 for other examples). The unit cost portion of the formula should be tied to the type of projects which will be undertaken (e.g., if infiltration is highly desirable in an area, the unit cost may be higher because of the generally higher cost associated with this type of project).

3) Determine the basic amount (of financial contribution) applicable to individual infill projects as they occur.

4) Assess the environmental benefit of site controls compared to the strategic application of funds to priority projects because in certain situations site controls may be required. Further, many commercial/industrial sites will benefit from the installation of on-site systems for dry weather flow and spills control.

5) Adjust the basic contribution amount by recognizing the costs of implementing site-specific controls. These costs could include engineering and design costs as well as materials and construction costs. In cases where policies require on-site control devices for specific types of land use (e.g., spill control devices), the value of on-site controls may be credited at a lower level.

6) Funds collected to implement priority projects.

5.4 Examples of Cost Calculation Methods

Various methods for calculating the amount of a financial contribution have been used in different jurisdictions. The Facility Cost and Area/Imperviousness methods are described below.

The approaches described have not been specifically assessed for compliance or compatibility with the *Development Charges Act*. Municipalities should work with their municipal solicitors to produce a legally sound development charge under the DCA.

In cases where a long-term stormwater management master plan and/or rehabilitation plan has not been completed, the Area/Imperviousness Method (5.4.2) is the most commonly used because of its simplicity and its link to the expected volume of water which will be generated. Depending on formulas used and local information, these methods can result in substantially different financial contribution requirements. Individual municipalities must, therefore, establish their own specific formulas to reflect equitable payments within the context of stormwater management facilities in their own area.

Municipalities periodically update cost factors used as better information becomes available and cost estimates change. In all cases, a municipality may choose to require on-site SWM controls rather than accepting financial contribution for off-site systems.

5.4.1 Facility Cost Method

Under the facility cost method, a SWMP design concept and cost is estimated based on similar projects. The required financial contribution is equal to the estimated cost of the facility (can exclude or include land costs). In most cases a wet pond concept is used for the estimate (even if this type of SWMP is not practical). For conditions where infiltration is practical, an infiltration trench design may be used for costing. The preparation of the cost estimate is normally the responsibility of the proponent, but is subject to municipal approval. The facility cost method has not been used extensively because of the effort required to establish the cost basis for facilities. However, some examples do exist.

City of Mississauga
The City of Mississauga has undertaken a number of studies and assessments to determine a Development Charges Levy for infill and "green fields" development. Mississauga now uses the Facility Cost Basis method to calculate financial contribution and links it to their Development Cost Levy. In developing the current approach, Mississauga identified the following:

- Erosion Control and Conveyance – Identified Projects;
- Erosion Control – Future Work;
- Stormwater Management Measures;
- Water Quality Control; and
- Future Oversizing.

Project costs were estimated for each category, and in the case of stormwater measures, land costs were also included (due to tableland placement of ponds). Based on this, a total development charges levy of $35,100/hectare ($21,700/gross hectare) was established.

5.4.2 Area/Imperviousness Method

Municipalities should be careful to ensure that any use of financial contribution models based on area/imperviousness method outlined below fit the rules and requirements of either:

(i) DCA regarding imposing a growth related SWM capital cost charges on new (re)developments; or

(ii) Planning Act regarding requiring a "payment in lieu of on-site SWM facilities" as a condition of a (re)development approval under that Act.

The area/imperviousness method is linked to the level of imperviousness, runoff, or contaminant-producing potential of the site. This is the simplest calculation method, and there are several variations that have been used by municipalities in Ontario.

Town of Markham
Most of the older parts of Markham are served by stormwater management facilities designed according to less stringent criteria when the areas were originally developed. Quality control was not practised before the 1990s; therefore, most of these areas are served by quantity control ponds which only minimize downstream flooding.

Older developed areas may offer opportunities for redevelopment or infill development that would be subject to current stormwater management standards for quantity, quality, and erosion controls. Although it may be possible to accommodate such controls on-site, there is the potential that on-site controls could lead to the proliferation of small and inefficient stormwater facilities.

The Town of Markham has developed an approach in conjunction with the Toronto and Region Conservation Authority (TRCA) to upgrade existing stormwater management facilities within the older parts of the Town to current standards (Town of Markham Stormwater Retrofit Study, 1999). Ten existing quantity ponds were identified and retrofitting costs ranged from $11,000 to $317,000 per pond for a total of $1,538,000. Existing uncontrolled storm sewer outfalls in the older parts of the Town were also assessed to determine the feasibility of constructing new facilities at these locations.

The TRCA commissioned a report entitled "Financial Contribution Toward Stormwater Controls" which provided estimates on the amount of money a development would have to contribute towards an off-site stormwater management project rather than undertaking on-site controls. The contribution was determined by undertaking a survey of costs for similar projects in the Greater Toronto Area and establishing an average cost per impervious hectare for developable land (Table 5.2).

Table 5.2: Average Unit Cost ($/Impervious Hectare (ha))

Pond Scenario	Construction ($)	10% Design and Review ($)	3% GST ($)	Total ($)
Quality Only	19,290.17	1,929.02	636.58	21,855.77
Quality and Quantity	25,645.40	2,564.54	846.30	29,056.24

City of Belleville

The City of Belleville estimates financial contribution (FC) based on the number of impervious hectares being developed and currently uses a factor amount of $10,000.

FC = (Area (ha) × % Imperviousness × Factor ($/ha)) + $10,000

6.0 OPERATION, MAINTENANCE AND MONITORING

6.1 History of Stormwater Management O & M

This chapter focuses on the operation and maintenance requirements for urban SWMPs. Monitoring and maintenance responsibilities are an important component of an effective SWMP and should be clearly defined in watershed/subwatershed plan implementation strategies. Maintenance and monitoring is also required for watershed/subwatershed projects such as natural channel designs and canopy cover restoration. However, this is not addressed in this manual.

During the 1970s and 1980s, stormwater management consisted of "peak shaving" facilities where peak flow under post-development conditions was reduced to that under pre-development conditions (2 year control for erosion, and up to 100 year for flooding). These facilities did not require sediment removal maintenance since the residence time of water within a peak shaving facility was in the order of several hours and there was marginal sediment/pollutant removal. Peak shaving facilities were designed to require as little maintenance as possible. However, regular maintenance is still required for inlet/outlet inspections, emergency spillway repair after a flood, trash removal, etc.

The introduction of stormwater management measures for water quality control has changed operations and maintenance needs. Many pollutants such as metals, bacteria, and nutrients bind to sediment. The design of urban water quality SWMPs is based primarily on sedimentation which requires sediment removal maintenance.

For stormwater facilities serving subdivisions, maintenance is the responsibility of the developer during the construction period (until works are assumed by the municipality). Stormwater facilities located on private property are the responsibility of the owner. In most cases, the municipality requires an easement agreement which specifies a required maintenance schedule and gives the municipality the right to enter the private property and conduct maintenance activities.

6.2 Importance of Maintenance

Maintenance is a necessary and important aspect of urban SWMPs design. One of the main reasons for SWMP failures and/or poor performance in the past was a lack of maintenance. Urban SWMP designers should give considerable thought to future long-term maintenance during the design of stormwater management practices.

In order to facilitate maintenance, it is advisable to prepare an annual maintenance report. The report should provide the following information annually:

- Observations resulting from inspection:
  - hydraulic operation of the facility (detention time, evidence or occurrence of overflows);
  - condition of vegetation in and around facility;
  - occurrence of obstructions at the inlet and outlet;
  - evidence of spills and oil/grease contamination; and
  - frequency of trash build-up.
- Measured sediment depths (where appropriate);
- Monitoring results, if flow or quality monitoring was undertaken;
- Maintenance and operation activities; and
- Recommendations for inspection and maintenance program for the coming year.

6.3 Operation and Maintenance Activities

There are many factors which influence sedimentation rates and maintenance requirements including: type of SWMP, land use, upstream development, and wildlife. Table 6.1 outlines operation and maintenance activities associated with different types of SWMPs.

Most SWMP monitoring has focussed on determining pollutant removal efficiency rather than maintenance/operations requirements of the facility. Since monitoring for maintenance is not common, the required frequency of maintenance activities is not well defined and activities tend to be performed on an "as required" basis.

One of the most important maintenance requirements for effective SWMP function is the removal of accumulated sediment which is discussed in Section 6.4. "The Storm Water Management Facility Sediment Maintenance Guide" (Greenland International Consulting Inc., 1999) provides additional information on sediment removal maintenance requirements.

Guidance on determining other maintenance requirements and frequency schedules is outlined in the following sections.

6.3.1 Inspections

SWM system inspections determine required maintenance activities. During the first two years of operation, inspections should be made after every significant storm to ensure proper functioning (average is about four inspections per year).

Table 6.1: Stormwater Management Practices Operation and Maintenance Activities

Item No.	Operation or Maintenance Activity	Type of Stormwater Management Practice											
		Wet Pond	Wetland	Dry Pond	Infiltration Basin	Infiltration Trench	Filter Strip	Superpipe Storage	Filters	Oil/Grit Separator	Soakaway Pit	Pervious Pipe	Grassed Swales
1	Inspection	■	■	■	■	■	■	■	■	■	■	■	■
2	Grass cutting	☐	☐	■	■	■			■				■
3	Weed Control	■	■	■	■		■		☐				■
4	Upland vegetation replanting	☐	☐	☐	☐	☐	☐		☐				
5	Shoreline Fringe and Flood Fringe vegetation replanting	☐	☐										
6	Aquatic vegetation replanting	☐	☐										
7	Removal of accumulated sediments	■	■	■	■	■	■		■	■		■**	■
8	Outlet valve adjustment	☐	☐	☐									
9	Roof leader filter cleaning/replacement										■		
10	Pervious pipe flushing											■	
11	Oil/Grit separator or Catchbasin cleaning									■		■	
12	Closing of infiltration facility inlet for winter months				■⊕	■⊕			■⊕			■⊕	
13	Trash removal	■	■	■	■	■	■	■	■	■*	■***	■	■
14	Infiltration basin floor tilling				■								

■ Normally Required ☐ May be Required

* *Litter removal part of sediment removal.*
** *Sediment removal part of catchbasin cleaning.*
*** *Litter removal by a filter in the rain gutter.*

⊕*Based on municipality experience and practices (e.g., may not be required if used on a local road with no salting or sanding).*

After this initial period, when the SWMP operation has been confirmed, annual inspections may suffice. A greater number of inspections may be required if the SWMP is poorly designed, or other factors such as upstream development which may cause operational or maintenance problems.

As shown in Table 6.1, regular inspections are required for all SWMPs including pre-treatment systems. Table 6.2 outlines some routine questions that could be used when inspecting various SWMPs.

6.3.2 Grass Cutting

Frequency
Generally, it is recommended that grass-cutting be limited or eliminated around SWM facilities since allowing grass to grow tends to enhance water quality and provide other benefits for wet facilities. Short grass around a wet stormwater facility provides an ideal habitat for nuisance species such as geese. Allowing the grass to grow is an effective means of discouraging geese.

Grass cutting is one maintenance activity which is solely undertaken to enhance the perceived aesthetics of the facility. The frequency of grass cutting depends on surrounding land uses, and local municipal by-laws. Therefore, grass cutting should be done as infrequently as possible, recognizing the aesthetic concerns of nearby residents.

Methods
Grass around wet facilities should not be cut to the edge of the permanent pool. As a safety precaution, cutting should be done parallel to the shoreline with grass clippings being ejected upland to reduce the potential for organic loadings to the pond.

6.3.3 Weed Control

Frequency
Weeds are generally defined as any kind of vegetation which is unwanted in a particular area. In terms of SWMPs, weeds are generally invasive species which cannot provide the intended function of the planting strategy, or non-native species such as purple loosestrife, the spread of which is undesirable. Weed control by-laws should be consulted for local requirements. Weed control may be required annually.

Methods
Weeding should be done by hand to prevent the destruction of surrounding vegetation. The use of herbicides and insecticides should be prohibited near SWMPs since they create water quality problems. The use of fertilizer should also be limited to minimize nutrient loadings to the downstream receiving waters.

Table 6.2: Potential Inspection Routine Questions for SWMPs

SWMP	Inspection Routine
Wet Ponds **Wetlands**	1. Is the pond level higher than the normal permanent pool elevation > 24 hours after a storm (or other design detention time)? (This could indicate blockage of the outlet by trash or sediment. Visually inspect the outlet structure for debris or blockage.) 2. Is the pond level lower than the normal permanent pool elevation? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.) 3. Is the vegetation around the pond unhealthy or dying? (This could indicate a poor selection of species. If occurs chronically further analysis should be conducted to identify the cause.) Is the pond all open water (no bulrushes or vegetation in the water)? Are there areas around the pond with easy access to open water? (This will indicate a need for replanting the pond) 4. Is there an oily sheen on the water near the inlet or outlet? Is the water frothy? Is there an unusual colouring to the water? (This may indicate the occurrence of an oil or industrial spill and the need for cleanup.) 5. Check the sediment depth in pond. (This will indicate the need for sediment removal. The sediment depth can be checked using a graduated pole with a flat plate attached to the bottom. A marker (pole, buoy) should be placed in the pond to indicate the spot(s) where a measurement should be made. A visual inspection on the pond depth can also be made if the pond is shallow and a graduated marker is located in the pond.)
Dry Ponds	1. Is there standing water in the pond > 24 hours after a storm (or other design detention time)? (This could indicate blockage of the outlet by trash or sediment. Visually inspect the outlet structure for debris or blockage.) 2. Is the pond always dry, or relatively dry within 24 hours of a storm (or other design detention time)? (This could indicate a blockage of the inlet or water quality/erosion control outlet which is too large. Visually inspect the inlet structure for debris or blockage.) 3. Is the vegetation around the pond unhealthy or dying? (This could indicate a poor selection of species. If occurs chronically further analysis should be conducted to identify the cause?) Are there areas around the pond with easy access to open water? (This will indicate a need for replanting the pond.) 4. Is there a visible accumulation of sediment in the bottom of the pond or around the high water line of the pond? (This will indicate the need for sediment removal.)
Infiltration **Basins**	1. Is there standing water in the basin > 24 hours after a storm? (This will indicate a decrease in the permeability of the underlying soils and, depending on the depth of water in the pond after 24 hours, the need for maintenance – sediment removal and roto-tilling of soils. If there is greater than one third the design depth of water in the pond 48 hours after a storm, the basin needs to be maintained.)

Table 6.2: Potential Inspection Routine Questions for SWMPs (cont'd)

SWMP	Inspection Routine
	2. Is the pond always dry, or relatively dry within 24 hours of a storm? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.) 3. Is there a visible accumulation of sediment in the bottom of the pond or around the high water line of the pond? (This will indicate the need for sediment removal.) 4. Are the top few inches of soil discoloured? (This may indicate a need for tilling of the soil.)
Infiltration Trenches	1. Is the trench draining? (Inspect the depth of water in the observation well. If the trench has not drained in 24 hours, the inlet and pre-treatment SWMPs should be cleaned (i.e., oil/grit separator, catchbasins, or grassed swales). If the trench has not drained within 48 hours the trench may need to be partially or wholly re-constructed to restore its performance. 2. Is the trench always dry, or relatively dry within 24 hours of a storm? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.)
Filter Strips	1. Are there areas of unhealthy, dead, or no vegetation downstream of the level spreader? (This will indicate the need to re-vegetate the filter strip.) 2. Are there indications of rill erosion downstream of the level spreader? (This will indicate the need to re-vegetate the filter strip. The rill erosion may be caused by a non-uniform spreader height. The spreader should be checked near the erosion areas to determine if it is in need of repair.) 3. Is there erosion of the level spreader? (The spreader should be re-constructed in areas where the spreader height is non uniform.) 4. Is there standing water upstream of the level spreader? (This will indicate that the level spreader is blocked. The level spreader should be checked for trash, debris, or sedimentation. The blockage should be removed and the spreader re-constructed if necessary.)
Buffer Strips	1. Are there areas of unhealthy or dead vegetation along the buffer strip? (This will indicate the need to re-vegetate the buffer strip.)
Filters	1. Are there areas of unhealthy or dead vegetation in a grass surfaced filter or a bioretention area? (This will indicate the need to re-vegetate the filter surface.) 2. Is there standing water in the filter > 24 hours after a storm? (This will indicate a blockage in the filter, possibly in the

Table 6.2: Potential Inspection Routine Questions for SWMPs (cont'd)

SWMP	Inspection Routine
	perforated pipe collection system or sedimentation on the surface or in the sand layer. The outlet collection system should be inspected for blockage. If there is water in the filter 48 hours after a storm, sediment removal should be undertaken. If sediment removal does not improve the performance (drainage) of the filter, the filter may need to be re-constructed.) 3. Is the filter always dry? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.) 4. Is there a visible discolouration of the top of the filter or accumulation of sediment on the filter? (This will indicate the need for sediment removal and/or replacement of the top few inches of the filter.)
Oil/Grit Separators (OGS)	1. Is there sediment in the separator/catchbasin? (The level of sediment should be measured using a graduated pole with a flat plate attached to the bottom. The pole should be graduated such that the true bottom of the separator/catchbain compared to the cover/grate is marked for comparison.) 2. Is there oil in the separator/catchbasin? (A visual inspection of the contents should be made from the surfacefor trash/debris and/or the presence of a oil/industrial spill. An oily sheen, frothing or unusual colouring to the water may inicate the occurrence of an oil or industrial spill. The separator/catchbasin should be cleaned in the event of spill contamination.)
Roof Leader Discharge to Soakaway Pits	1. Are there frequent overflows to the surface during small storm events? (Frequent overflows will indicate tha roof leader filter has clogged or the soakaway storage media has become clogged. The filter should be checked for an accumlation of leaves and twigs. If the filter is clean, the pit may need to be reconstructed to restore its performance.)
Perforated Pipe Systems	1. Are the pre-treatment SWMPs operating properly? (Pre-treatment SWMPs should be inspected (see oil/grit separators, grassed swales).) 2. Is the perforated pipe operating properly? (The connection to the perforated pipe (i.e., manhole/catchbasin) should be visually inspected for standing water 24 hours after a storm. Standing water will indicate the need for maintenance ofthe perforated pipe system (flushing, jet washing).)
Grassed Swales	1. Is there standing water in an enhanced grass swale. (This will indicate a blocked check dam or decrease in the permeability of the swale. The check dam should be inspected for blockage by trash/debris, or sediment.) 2. Is the grass/vegetation unhealthy or dead? (This will indicate the need to revegetate the swale.) 3. Is there erosion downstream of the swale? (This may indicate frequent overtopping of the swale, and as such blockage of the dam or decreased swale permeability. The dam should be inspected for blockage and the erosion corrected by sodding. There may be a need to provide further erosion control (rip-rap, plant stakings) to prevent the re-occurrence of erosion.)

6.3.4 Plantings

Frequency

Upland and flood fringe plantings are generally stable and should not need much maintenance or re-establishment. Shoreline fringe areas are subject to harsher conditions as a result of the frequent wetting and drying associated with this zone. Aquatic plantings are the hardest to initially establish. It is anticipated that vegetation in the aquatic and shoreline fringe zones will require some replanting or enhancement during the first two years of SWM facility operation. Preliminary results of stormwater plantings studies indicate that a healthy vegetative community will establish if proper conditions are created (although the final set of species will often not be those that were originally planted).

Methods

Table 6.1 outlines SWMPs to which planting of vegetation applies. Planting methods can be separated into three main categories based on the wetness level and types of vegetation that will grow in these conditions (terrestrial to aquatic):

Upland/Flood Fringe

The two types of plantings used are ground cover (grasses, herbs) and woody shrubs and trees. Planting should occur in the spring after water levels have subsided to a stable level. Ground cover could be installed either by hydroseeding or using a custom seed mix in a nutrient rich medium impregnated in a biodegradable mesh-like blanket. Individual shrubs and trees could be planted manually with openings made in the blanket for each planting if necessary.

Shoreline Fringe (Wet Riparian)

Shoreline fringe plantings should be carried out in mid-May to early June but after water levels have subsided to a stable level. Some form of protection of the seed mixture and soil nutrient medium (if required) should be provided in this dynamic zone of water level fluctuation. The biodegradable mesh-like blanket suggested in the upland zone is also highly recommended in this zone to establish ground cover. Shrubs and trees can be planted through openings created in the blanket.

Aquatic Fringe/Shallow Water

The establishment of plantings in this zone will require greater materials handling and growth monitoring both in the short and the long-term. Emergent vegetation is easily planted by hand if the substrate is suitable. Ideally, a firm substrate with at least 10% organics (by volume) allows emergent vegetation to be hand planted. Young shoots, as opposed to rhizomes or corms, are preferable for planting as these plants are already growing with an established root structure (for early stability). The plants should be at least 10 cm tall and planting should occur in late May to early June.

Sprigs or plugs of emergent plant material are preferable for planting since root material is already contained in a suitable growth medium.

Submerged rooted plants (including pondweeds) should be planted as mature vegetative growth if planted in late spring to early summer. Mature growth will take advantage of warmer water and sunlight penetration. Plantings in early spring or fall should use vegetative propagules such as turions or rhizome plugs which can germinate in the spring or over the winter and begin growing in the following growing season.

6.3.5 Outlet Valve Adjustment

Extended detention outlets should be designed to allow for the adjustment of detention times. Information about the effects of detention times on water quality enhancement, erosion, and flooding is still evolving and there may be a need for operational changes in the field to address site-specific or subwatershed related concerns on a case-by-case basis (especially when subwatershed planning has not been undertaken).

6.3.6 Trash Removal

Trash removal is an integral part of SWMP maintenance. Generally, a "spring cleanup" is needed to remove trash from all surface SWMPs. Trash removal is then performed as required based on observations during regular inspections.

6.4 Sediment Removal Maintenance Issues

6.4.1 Frequency Removal

To ensure long-term effectiveness, the sediment that accumulates in SWMPs (e.g., wet ponds, wetlands and dry ponds) should be periodically removed. The required frequency of sediment removal is dependent on many factors including:

- type of SWMP;
- design storage volume (e.g., if active and permanent pool storage is oversized for sediment storage);
- characteristics of the upstream catchment area (e.g., land use; level of imperviousness; upstream construction activities and effectiveness of sediment and erosion control activities); and
- municipal practices (e.g., sanding).

There is limited data available on sediment accumulation. Monitoring of new ponds and retrofit ponds (converted ponds in older established areas) indicates a significant difference in sediment buildup for different ponds at different time periods. Sediment accumulation will typically be rapid for the entire construction period (including time required for the building, sodding and landscaping of individual lots). Once a catchment area is completely developed and vegetation is established, sediment accumulation drops markedly.

Continuous simulations were performed for end-of-pipe stormwater management facilities to assess the rate of sediment accumulation (see Section 3.3.1). The continuous simulation indicated total suspended solid (TSS) removal efficiencies for different end-of-pipe stormwater management facilities with varying volumes of storage and different levels of imperviousness. The removal efficiencies were converted into volumes of sediment captured by each type of facility on an annual basis. A set of curves was developed which indicate sediment removal frequency for facility type, storage, and level of upstream imperviousness (Figures 6.1 to 6.4).

Sediment accumulation reduces the effective storage volume and the long-term SWMP removal efficiency of suspended solids. The theoretical maintenance frequency for sediment removal can be calculated based on the rate of performance reduction with loss in storage volume. The theoretical performance-storage relationship does not account for conditions such as upstream development and poor sediment and erosion controls. As a result, these maintenance frequencies are only estimates which should be refined based on operational and maintenance experience in the field.

The performance-storage curve becomes asymptotic quickly (a large increase in storage is required for small improvements in the removal performance). This means that for typical SWMP storage volumes there must be a considerable loss in storage to reduce the effectiveness of the facility. It was assumed that 5% was an acceptable reduction in TSS removal efficiency due to gradual sediment accumulation.

The average annual TSS removal efficiency of a SWMP with a certain volume of storage was determined using continuous simulation and a sedimentation model (see Table 3.2 – Chapter 3). The required maintenance frequency for this SWMP was then determined based on the annual sediment accumulation and resulting annual loss in storage. The timeframe to reduce the storage to the point that the annual removal efficiency was 5% less than the original efficiency indicates the maintenance frequency for that SWMP with that particular storage.

If excess storage is provided to lengthen the intervals between required maintenance, the timeframe to reduce the efficiency by 5% below the original efficiency should be calculated. For example, if 80% removal is required, but excess storage is provided resulting in a initial efficiency of 85%, then maintenance would be required when the performance efficiency was reduced by 10% (i.e., 5% below the original target efficiency).

Curves of maintenance frequency by SWMP type, storage, and different levels of upstream imperviousness were calculated based on the continuous simulation results and the requirement for maintenance with a 5% loss in TSS removal performance (Figures 6.1 to 6.4). These curves are best-fit lines based on linear regression over a period of 50 years and indicate that there is a linear relationship between maintenance frequency and SWMP storage.

Figure 6.1: Storage Volume vs. Removal Frequency – for 35% Impervious Catchments

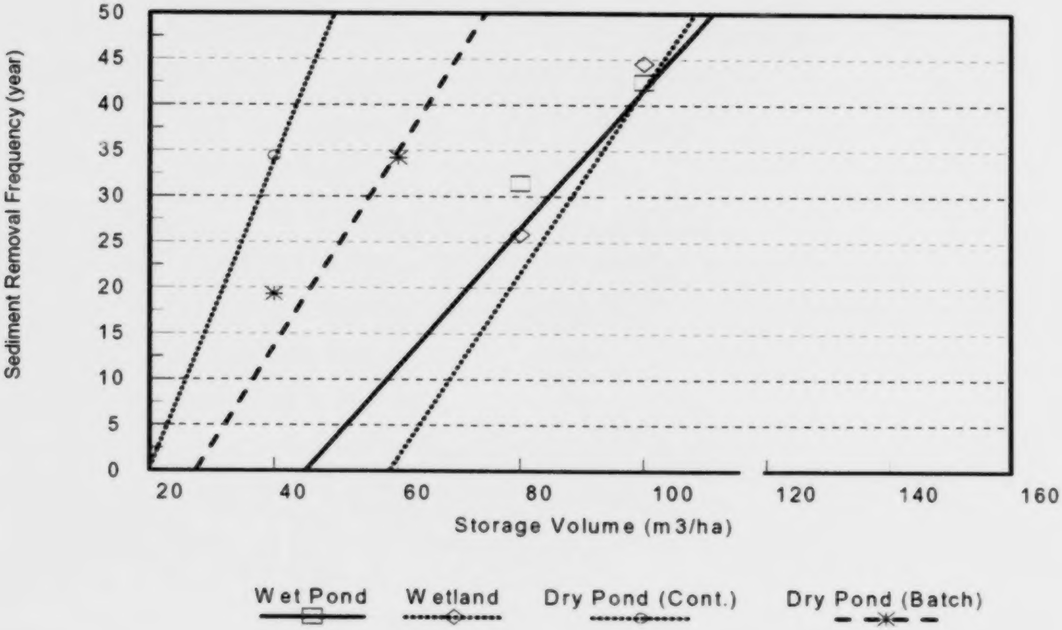


Figure 6.2: Storage Volume vs. Removal Frequency – for 55% Impervious Catchments

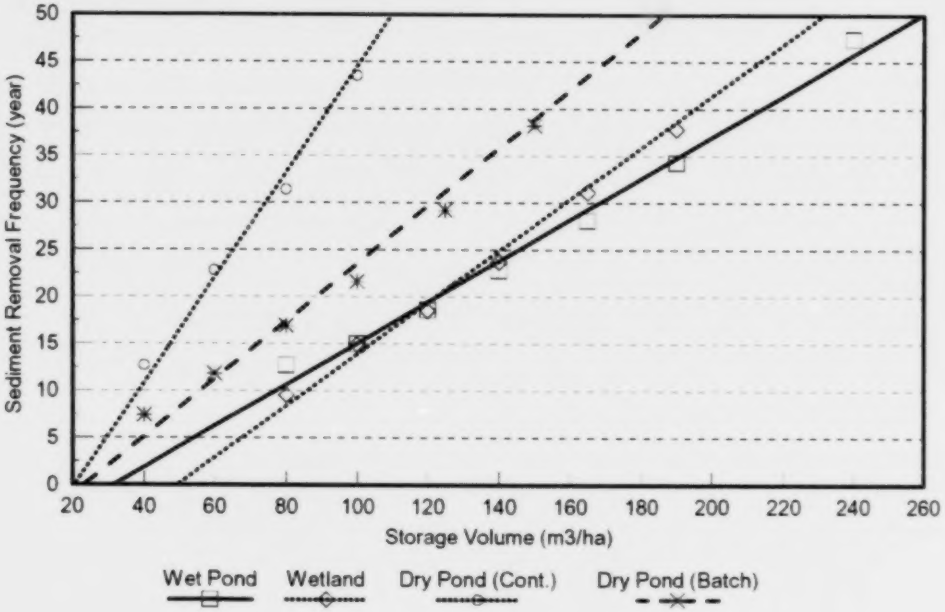


Figure 6.3: Storage Volume vs. Removal Frequency – for 70% Impervious Catchments

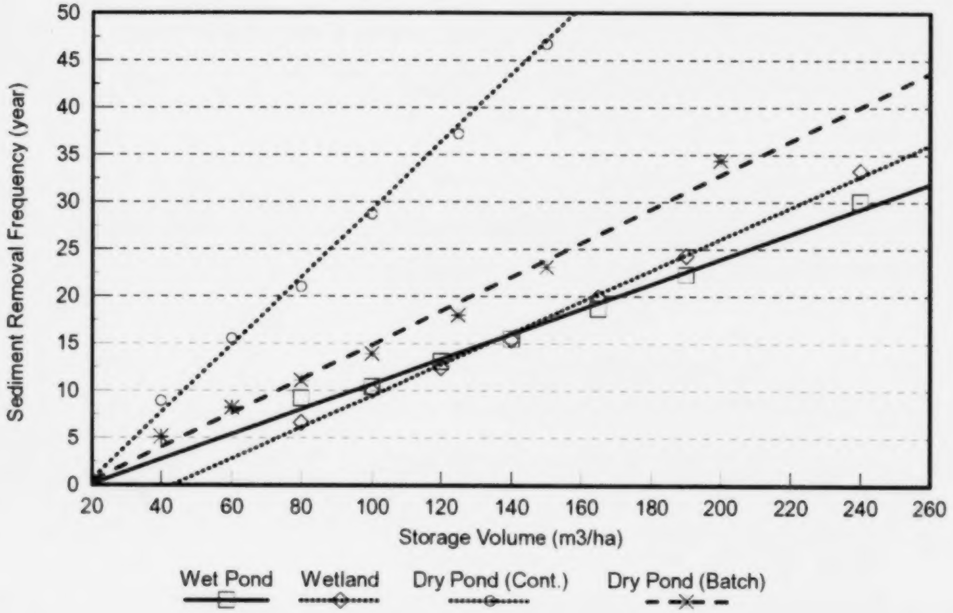
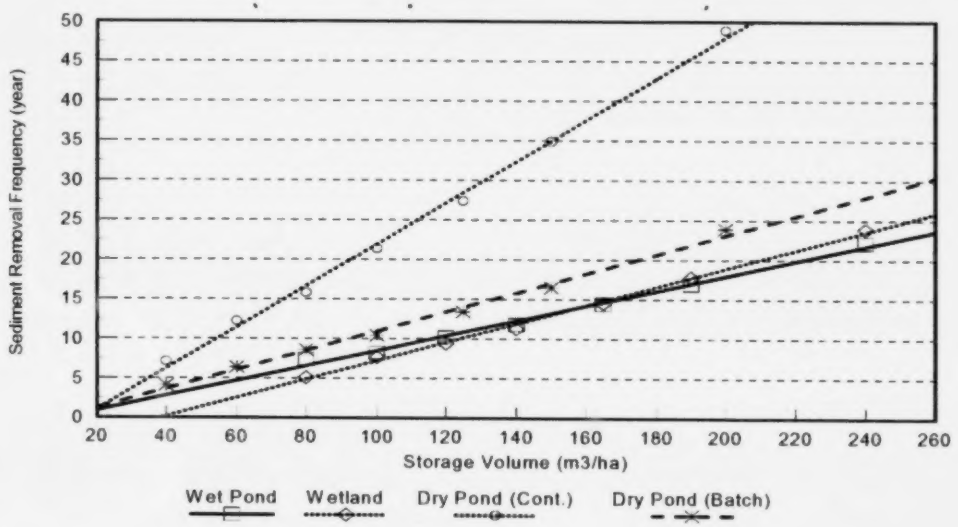


Figure 6.4: Storage Volume vs. Removal Frequency – for 85% Impervious Catchments



These graphs can be used to determine the required sediment removal frequency given the SWMP type, storage volume, and imperviousness level of catchment basin.

Figures 6.1 to 6.4 also indicate that increased storage capacity increases the maintenance interval (i.e., less frequent maintenance required). These curves are based on the assumption of 5% loss of performance and should not be used for over-sized facilities. In order to allow users to calculate the required maintenance frequency for an oversized SWMP, annual suspended solids loadings in runoff from catchments with different levels of imperviousness and estimated sediment density are provided in Table 6.3.

The values of suspended solids loadings in Table 6.3 were derived from US Environmental Protection Agency (EPA) Stormwater Management Model (SWMM) simulation results and are only intended to be used as estimates for planning purposes. The density of suspended solids was based on a review of the literature of stormwater sediment characteristics and recent pond sediment removal data. The following methodology should be used to calculate the maintenance frequency if storage for the SWMP is oversized (calculation can be easily automated in a spreadsheet format):

1. Determine the appropriate total suspended solid (TSS) removal efficiency based on level of protection required for receiving stream (See Table 3.2).

2. Subtract 5% to obtain the target maintenance removal efficiency.

3. Determine the projected TSS removal efficiency based on the storage provided.

4. Calculate the loss in removal performance and loss in storage for each year based on the removal performance at the start of the year, the suspended solids loading rate, and the sediment density. The removal efficiency at the start of the next year will be based on the resulting available storage volume at the end of the year. These calculations are continued until the removal efficiency of the facility at the start of the year is equal to the target maintenance removal efficiency.

Table 6.3: Annual Sediment Loadings

Catchment Imperviousness	Annual Loading (kg/ha)	Wet Density (kg/m^3)	Annual Loading (m^3/ha)
35%	770	1,230	0.6
55%	2,300	1,230	1.9
70%	3,495	1,230	2.8
85%	4,680	1,230	3.8

Alternatively, a conservative estimate of annual sediment accumulation in a SWMP may be obtained by multiplying the annual loading of suspended solids (m^3/yr) (see Table 6.3) by the initial removal efficiency for the particular SWMP. Using this method, a calculation is made to determine how long it takes to accumulate the difference in storage volumes between the initial storage and the target maintenance storage volume.

6.4.2 Methods

The methods for sediment removal depend on the type of SWMP implemented. The following sections describe sediment removal techniques for different types of SWMPs.

6.4.2.1 Soakaway Pits

Soakaway pits should only be used to infiltrate relatively clean water (rooftops, pervious areas) to reduce the potential for clogging. The roof leader discharge system should have an overflow discharge to the surface and be as close to the ground as possible to minimize the build up of head on the soakaway pit. A plastic mesh or wire mesh filter should be placed near the ground surface just below the overflow pipe so that overflows will occur if the filter becomes plugged. Frequent overflows during small summer storms are a signal that maintenance is required. The filter should be cleaned once a year in the fall (after the leaves have fallen off the trees).

6.4.2.2 Grassed Swales

Visual inspection and the aesthetic attributes of swales will indicate the need for maintenance. In areas which receive road runoff, discolouration of the soils or the build-up of a "crust" may indicate the need for mulch (to maintain infiltration properties).

6.4.2.3 Pervious Pipe Systems

It should be recognized that relatively little is known about maintenance requirements for pervious pipes conveying stormwater since maintenance techniques have not been extensively tested. Monitoring is required to provide a better understanding of the effectiveness (if any) of various techniques. The City of Toronto (Etobicoke) was monitored for a short time and has shown no indications of clogging. A system which has been in place in Ottawa for some time has also performed well.

Pre-treatment of the stormwater before it enters the perforated pipe system is fundamental to the longevity of the system. Maintenance activities relating to the pre-treatment system (grassed swales, oversized catchbasins, street sweeping, manhole oil/grit separators), and source control measures (salting and sanding practices) should be implemented to minimize the volume of particulate matter conveyed to the perforated pipe system.

In addition, the feasibility of seasonal operation of the system should be investigated since most road systems in Ontario require frequent sanding and salting during the winter. Winter operation may degrade groundwater quality and decrease the lifespan of the system by clogging the pipe perforations and the void spaces in the surrounding backfill/storage material.

Catchbasin cleaning is done via the use of a vacuum truck which extends a hose into the sump of the catchbasin and sucks out the material which has been deposited in the sump. As a general rule, catchbasin cleaning should be done annually; however, municipalities should adjust the frequency based on the volume of material removed.

The following three maintenance techniques for storm sewers and leachate collection systems may be applicable to the maintenance of perforated pipe systems; however, further monitoring is required to determine their effectiveness:

Flushing
Sewer flushing is generally undertaken to clean out material which has been deposited in the pipe. A filter sock may be used to prevent fine material from entering the pipe system from the native material. However, this could lead to clogging at the pipe/filter cloth interface which sewer flushing may not be able to remedy. This should be considered in the design of the system.

Radial Washing
Radial washing is similar in operation to flushing. The perforated pipe must be connected between manholes and the downstream end plugged or capped. A water hose is connected to the upstream end of the perforated pipe and water is introduced from the surface into the hose. The perforated pipe is essentially pressurized, forcing water out the perforations and hence, cleaning plugged perforations. Radial washing can be performed after flushing if there is considerable sediment deposition in the pipe itself.

Jet Flushing
Jet flushing is frequently used in leachate collection systems for landfills to clean the perforated collection pipes. A pressurized hose is attached to an end nozzle which discharges water in various directions to clean the pipe. The pressure in the pipe on the end nozzle also directs the hose further along the pipe (i.e., self directing). There are various nozzle designs available, and one which directs water radially into the perforations would be appropriate for perforated storm sewer applications.

6.4.2.4 Infiltration Trench

Maintenance of these systems generally focuses on ensuring that the pre-treatment is operational and adequate. Flushing of pipes in an infiltration trench is generally not feasible since there are typically several pervious pipes within the trench and cleanout locations would have to be provided at both the inlet and outlet of each pipe length. In addition, flushing may not be effective as discussed previously (Section 6.4.2.3). Other than maintaining pre-treatment measures, the only feasible maintenance for infiltration trenches is re-construction.

6.4.2.5 Infiltration Basin

Infiltration basins are end-of-pipe stormwater management facilities with highly permeable soils. Accumulation of sediment in these surface storage facilities seals the bottom and reduces the ability of soil to allow infiltration of stormwater. Maintenance of pre-treatment SWMPs and the implementation of source controls (salting and sanding practices) will help prevent sediment build-up.

Pre-treatment will not be totally effective in preventing suspended solids from entering an infiltration basin. Tilling the land may be required to maintain the infiltration potential of the soil. In areas where tilling has been tried, little success has been achieved in maintaining the infiltration potential (Maryland, 1993). Experience indicates that deep tilling must be employed (i.e., a rototiller will not dig deep enough). Planting of deep rooted legumes in an infiltration basin may be beneficial in maintaining the porosity and infiltration potential of the soil. However, consideration must be given to the anticipated growing conditions in the basin (frequency and depth of inundation). Deep basins (> 0.6 m) are not recommended since the weight of water tends to compact the soil. Once an infiltration basin has sealed, remediation is difficult and expensive and may not even be successful.

6.4.2.6 Filters

Filters can either be surface or subsurface end-of-pipe stormwater management facilities. Surface filters may or may not have a grass cover. Filters without a grass cover can be raked to prevent clogging and to remove trash. Maintenance requirements for grass-covered filters, bioretention areas, or subsurface filters are similar to those for infiltration trenches and should focus on ensuring that adequate pre-treatment and source controls are provided.

6.4.2.7 Vegetated Filter Strips

Maintenance activities for filter strips involve removing sediment from upstream of the level spreader, ensuring that the level spreader is operating in accordance with the design and maintaining the vegetated strip to promote sheet flow. Sediment removal from upstream of the filter strip can be done using a vacuum truck or small grading equipment if there is considerable sedimentation. Given the small drainage areas serviced by these SWMPs, the volume of sediment to remove will be limited.

6.4.2.8 Buffer Strips

Buffer strips are generally not engineered and will not provide any location for concentrated collection of sediment. Sediment removal is not proposed since the sediment will be dispersed and removal would destroy the vegetation and the primary buffering capacity in this area.

6.4.2.9 Oil/Grit Separators

Manhole oil/grit separators (OGS) should be cleaned out using a vacuum truck. Some interceptors discharge low flows containing oil and grit to the sanitary sewer. Although this type of design facilitates maintenance, it is undesirable in the case of a large fuel/oil spill since the sewage treatment plant cannot treat large loadings of these pollutants. Therefore, it is recommended that any outlet to the sanitary sewer from the oil/grit separator be valved and kept closed during everyday operations. Manhole separators or three-chamber separators that incorporate a by-pass should be cleaned out annually and after any known spills have occurred.

6.4.2.10 Wet Ponds, Dry Ponds, and Wetlands

Typical grading/excavation equipment such as backhoes and in some instances hydraulic dredging should be used to remove sediment from ponds and wetlands. Certain types of backhoes and loaders have a tendency to tear up the inter-locking block on the hardened floor. Therefore, there has been a shift to using long-reach backhoes. Conventional dredging is not recommended because of the costs and potential to destroy features in the facility (i.e., vegetation and bottom grading).

Regardless of the means selected for sediment removal, the procedure should meet the requirements normally imposed by a sediment and erosion control plan (e.g., no off-site migration of sediment to roads, stormwater conveyance systems or watercourses).

Theoretical sediment removal frequencies for these SWMPs are provided in Section 6.5.

6.4.3 Sediment Disposal

Generally, sediment removed from SWMPs will not be contaminated to the point that it would be classified as hazardous waste. However, all sediment which is removed from SWMPs should be tested to determine disposal options. MOE sediment disposal requirements should be consulted for information pertaining to the exact parameters and acceptable levels for different disposal options. Most private laboratories are familiar with the disposal guidelines and can test sediment samples with these in mind.

For example, in order to deposit the sediment on land, it would need to meet inert fill requirements under Regulation 347. For landfill disposal, the sediment would have to be classified as non-hazardous, i.e., not leachate toxic according to TCLP leachate test in Regulation 347 (effective March 31, 2001).

There are three generalized disposal options:

On-Site Disposal
On-site disposal allows the sediment to be disposed of on any land area that is not regulated (i.e., land other than floodplain, etc.). In the planning stage of land requirements for subdivision/site plan stormwater management requirements, land can be set aside for on-site disposal of sediments to be removed from the various SWMPs. The areas that are used for sediment disposal should be landscaped to provide a natural appearance after each sediment removal operation.

Off-Site Disposal
It is anticipated that off-site disposal may be preferred by most developers and municipalities since off-site disposal does not reduce the developable area, landscaping/grading does not have to be performed, and there are no perceived liability/health concerns with respect to the surrounding landowners. Off-site disposal can mean disposal at a sanitary landfill or disposal at another area undergoing filling. The decision of where the material is deposited depends on the quality of the sediments and the availability and distance of the alternative fill areas.

Temporary disposal areas are recommended for surface end-of-pipe stormwater management facilities particularly those that do not have a maintenance by-pass since it provides a location for the sediment to dry before transporting it off-site. Where temporary sediment disposal areas (i.e., drying areas) are not feasible due to limited availability of land or high cost, the means of dealing with the un-dewatered sediment should be detailed in the stormwater management plan and approved by the municipality.

Hazardous Waste Disposal
Although sediment removed from SWMPs is expected to contain contaminants (metals, bacteria, nutrients), it will not likely be classified as hazardous waste. Hazardous waste must be deposited at a hazardous waste facility. Transportation costs and disposal fees are expensive for hazardous waste since licensed haulers must be used to transport the material and there are relatively few facilities in the province.

6.5 Winter Operation

Section 4.3 describes general SWMP design modifications which should be considered in cold climates and section 4.5 specific considerations for different types of SWMPs. These sections address both the issue of reduced performance and the susceptibility of SWMPs to damage that may result from cold temperatures.

Infiltration facilities are subject to reductions in capacity due to freezing or saturation of the soil. Surface filters and bioretention areas are generally subject to similar problems. Subsurface filters, while less susceptible than surface filters, may demonstrate poorer performance in the winter due to freezing in underdrain pipes or the filter medium. Filters which utilize organic medium are particularly prone to freezing because they retain water.

There is an increased likelihood of clogging of infiltration facilities and filters during winter operation due to the high sediment loads resulting from road maintenance activities (e.g., sanding and salting). There is an increased risk of groundwater contamination from road salt associated with winter operation of infiltration facilities that receive road runoff.

To prevent groundwater contamination and damage of water quality SWMPs may be activated several weeks before the average annual date of the first frost and deactivated in the spring when snowmelt is complete. In areas with curb and gutter servicing, the road system should be swept before by-passes are deactivated.

In most cases, filters and infiltration systems are part of a treatment train such that runoff which by-passes these SWMPs will still pass through downstream controls.

6.6 Maintenance Enhancements

It is important that SWMP planners and designers consider maintenance activities in their design (see Chapter 4 for further details) including:

Access
A maintenance route should be established to allow vehicles access to SWMP. The slope of the access route should accommodate maintenance vehicles (i.e., 4:1 or flatter). Access to

stormwater lot level controls may not be possible given the tendency for homeowners to construct fences, gardens, landscaping, etc. If stormwater lot level or conveyance controls (i.e., enhanced swales or trenches) are proposed along rear lot lines, municipalities can obtain an easement for maintenance. The logistics of maintaining access to the easement will require considerable diligence/effort on the part of the municipality and may not be feasible.

Access to inlet and outlet structures, flow splitters, and by-pass manholes/chambers is also important. Access to an outlet structure for a pond or wetland can be provided by placing the outlet in a chamber in the embankment. Locating the outlet in a chamber enhances the aesthetics of the SWMP and reduces the potential for vandalism.

Forebays

Forebays are applicable for most end-of-pipe stormwater management facilities (wetlands, wet ponds, dry ponds, infiltration basins). Forebays allow the sediment deposits to be concentrated in one location thereby facilitating maintenance operations. To minimize the potential for scour and resuspension, forebays may have a deep permanent pool which should be drawn down for maintenance.

If water will remain in the downstream portion of the facility during maintenance, the berm between the forebay and the rest of the facility will need to be designed as a small dam. In cases where the forebay releases to a dry pond or infiltration basin, a gravity drainable pipe (if physically feasible) can be installed in the berm to draw down the forebay.

In cases where the forebay releases to a wet pond or wetland, there are two options. The water level in the downstream portion of the facility can be lowered until the berm is emergent. Water can then be pumped from the forebay to the downstream portion of the facility until the forebay is dry. Maintaining water in the downstream portion of the facility has the benefit of reducing the impacts to the aquatic and shoreline fringe vegetation. The second option would be to drain both facilities. This could be accomplished by either valved gravity draining maintenance pipes (if feasible) in both the forebay and the downstream portion of the facility, or by pumping if the facilities cannot be gravity drained.

Maintenance/Drawdown pipe

Maintenance pipe should be provided to draw down a forebay's permanent pool for maintenance. This maintenance pipe should be set near or at the bottom of the facility. If gravity drainage is not feasible, the facility will have to be pumped when maintenance is required. If possible, the pond should be drawn down early in the morning or overnight to reduce downstream thermal impacts. A geotextile filter bag should be attached to the end of the maintenance pipe to prevent the discharge of sediment from the facility into the receiving waters.

Pre-treatment

Adequate pre-treatment (oil/grit separators, roof leader filter traps, grassed swales) should be provided for infiltration or filtration SWMPs. These measures are described in Chapter 4.

Maintenance By-pass
Maintenance may take from several days to a week to perform. Storms during this time should be routed around the SWMP (see section 4.6). The by-pass should be located either at the inlet or slightly upstream of the SWMP. In piped systems, this is accommodated by fitting sluice gates to the by-pass pipe and SWMP inlet pipe in an upstream manhole. For maintenance operations, the gate to the SWMP can be closed and the gate to the by-pass pipe opened. This type of system can also be used for the seasonal operation of infiltration systems that accept road runoff.

Over-Sizing SWMP Storage
Over-sizing the storage provided in a SWMP compared to what is required to achieve performance targets will decrease the maintenance frequency in a SWMP. It is left to the discretion of municipalities to increase volume requirements for reduced maintenance frequency (if desired) beyond provincial/municipal water management requirements.

Sediment Disposal Areas
Where adequate land is available, sediment removal operations and costs can be reduced if an area is set aside for sediment disposal (e.g., when a stormwater pond is paired with a public park). These areas can be used for either permanent sediment disposal or temporary disposal (to allow the sediment to dry before transporting off-site for permanent disposal). Temporary drying areas are recommended for surface end-of-pipe stormwater management facilities that do not have a maintenance by-pass.

6.7 Monitoring

Stormwater monitoring is typically conducted at two levels:

Watershed and Subwatershed Monitoring
As noted previously, stormwater is best managed within the context of a watershed and subwatershed plan. These plans will normally contain a monitoring component to track implementation of the plan. The monitoring program will typically include administrative monitoring, water chemistry, biological monitoring, flow and erosion monitoring. These monitoring programs are essential to the success of the Plan. Subwatershed monitoring will normally be conducted or administered by the local conservation authority or municipality.

Facility Monitoring
The consensus of opinion among practitioners is that monitoring for chemistry or biotic parameters cannot be justified for each individual facility because to have any scientific validity a large and costly sampling program is required. The approach generally used within the province is physical operation monitoring by the proponent to verify that the facility is operating as designed and detailed pilot site monitoring through research programs to evaluate effectiveness issues. The designer is advised to consult with authorities regarding site-specific requirements because some jurisdictions have additional monitoring requirements.

7.0 CAPITAL AND OPERATIONAL COSTS

7.1 Costing Information

This chapter provides information on capital as well as operation and maintenance costs for stormwater management practices (SWMPs). The cost of urban stormwater solutions is an important component to the overall economic viability of a development and needs to be considered when assessing alternative stormwater management measures. Maintenance and operation costs are critical considerations in the overall costs since these facilities will need to be maintained and operated in perpetuity.

Information outlined is based on 1997 to 1998 construction costs in the Greater Toronto Area and should only be used for planning purposes. Site-specific costs of SWMPs should be determined in all cases since there are many site-specific conditions that will affect capital, operation and maintenance costs.

SWP planning involves investigating SWMP alternatives at the site, subdivision, or subwatershed level. The information in this chapter can be used to estimate costs for various SWMP solutions (alone and in combination). This information will likely be most useful at a plan of subdivision level and subwatershed level since site-specific information will probably be available at the site plan level to provide more accurate costing estimates.

The total cost of a SWMP includes capital costs, the present value of operation and maintenance costs, engineering costs and contingency costs. Each of these costs is discussed in subsequent sections of this chapter.

7.2 SWMP Capital Costs

Stormwater facility capital costs are based on estimated construction costs including: excavation; cutting and filling; grading; structures; fittings; environmental site controls (sediment and erosion controls); and material costs. Engineering and design costs as well as land costs could also be included.

Table 7.1 outlines the type of construction and materials that may be required for various end-of-pipe stormwater management facilities. As noted in Table 7.1, several variations of items (e.g., reverse sloped outlet pipe, riser outlet pipe) have been included for different SWMP configurations. Therefore, when doing costs estimates, duplicate items in Table 7.1 should not be included twice (i.e., cost for a reverse sloped outlet pipe or a perforated riser outlet pipe, but not both).

Table 7.1: Capital Cost Items for End-of-Pipe Stormwater Management Facilities

Type of Construction or Material	wet pond	wetland	dry pond	infiltration basin	infiltration trench	soakaway pit	filter strip	sand filter	oil/grit* separator	grass swales	flow splitter
Excavation (off-site disposal)	×	×	×	×	×	×	×	×		×	×
Earthwork (cut and fill on site)	×	×	×	×	×	×	×	×		×	×
Erosion block/stone	×	×	×								
Concrete Outlet Structure	×	×	×								
Concrete Outlet Pipe	×	×	×								
Perforated Riser Outlet	×	×	×								
Perforated Riser Outlet Trash Rack	×	×	×								
Infiltration Observation Well					×						
Rip-rap	×	×	×							×	
Perforated Pipe				×	×	×	×	×			
Seed and Topsoil	×	×			×						
Clear Stone (gravel)					×	×		×			
Filter Cloth					×	×		×			
Filter Material (sand)					×			×			

Table 7.1: Capital Cost Items for End-of-Pipe Stormwater Management Facilities (cont'd)

Type of Construction or Material	wet pond	wetland	dry pond	infiltration basin	infiltration trench	soakaway pit	filter strip	sand filter	oil/grit* separator	grass swales	flow splitter
Submergent and Emergent Vegetation	×	×									
Shoreline Fringe and Flood Fringe Vegetation	×	×	×								
Upland Vegetation	×	×	×	×			×				
Temporary Fencing (post and wire)	×	×									
Grass Sod and Topsoil			×	×			×	×		×	
Concrete (poured in place)									×		×
Trash Rack (metal)									×		
Inverted Elbow Pipe									×		
Outlet Valve/Gate Controls	×	×	×								×

× usually required

* *3 chamber oil/grit separator.*

Note: Several variations of items (e.g., reverse sloped outlet pipe, riser outlet pipe) have been included for different configurations. Therefore, when doing costs estimates, duplicate items should not be included twice (i.e., cost for a reverse sloped outlet pipe **or** a perforated riser outlet pipe, not both).

7.2.1 Pre-Treatment SWMPs

Some of the SWMPs listed in Table 7.1 require pre-treatment to ensure proper operation and longevity. Pre-treatment is required for infiltration SWMPs to reduce the potential for clogging and to avoid the deterioration of groundwater quality.

Storage (wet pond, wetland, dry pond) and/or vegetative SWM pre-treatment practices are generally used upstream of the SWMP requiring protection. To a certain extent the size of pre-treatment SWMP will depend on land availability. However, the size can be estimated using the sizing rules provided in Chapter 4. The size of the storage SWMPs should be based on the forebay sizing rules and not on providing full water quality treatment. Table 7.2 provides a list of SWMPs that require pre-treatment and possible pre-treatment SWMPs. In the case of surface end-of-pipe stormwater management facilities, forebays can be considered pre-treatment.

Table 7.2: Pre-Treatment SWMPs

		Pre-Treatment SWMPs				
SWMP	**Need for Pre-Treatment**	**Grassed Swales**	**Filter Screen**	**Filter Strip**	**Oil/Grit Separator**	**Wet Pond**
Wet Pond	⊠	×				×
Wetland	⊠	×				×
Dry Pond	⊠	×				×
Infiltration Basin	■	×		×	×	×
Infiltration Trench	■	×		×	×	×
Pervious Pipe	■	×			×	
Pervious Catchbasin	■	×				
Soakaway Pit	■		×			
Filter Strip	⊠	×				×
Sand Filter	⊠	×		×	×	×
Oil/Grit Separator	□					
Grassed Swales	□					

■ Pre-treatment required

⊠ Pre-treatment enhances performance

□ Pre-treatment not required

Table 7.3 lists unit prices for different types of construction activities and material associated with the SWMP. These unit price estimates are for normal construction circumstances and include labour. Local or site and project-specific estimates should be made whenever possible.

Table 7.3: Unit Costs for Capital Construction

Type of Construction or Material	Unit	Price
Land Alterations		
Excavation (off-site disposal)	m^3	$ 5 - 10
Earthwork (cut and fill on site)	m^3	$ 3
Construction Materials		
Erosion block/stone	m^2	$ 50
Concrete Outlet Structure	each	$ 5,500
Concrete Outlet Pipe (300 mm / 600 mm / 900 mm)	m	$ 70 / $ 170 / $ 300
Observation Well (100 mm PVC)	each	$ 15
Rip-rap (450 mm)	m^2	$ 50
Perforated Pipe (100 mm, plastic)	m	$ 10
Perforated Riser Outlet Pipe (300 mm, plastic)	m	$ 90
Perforated Riser Outlet Trash Rack (400 mm CMP)	m	$ 100
Temporary Fencing (post and wire)	m	$ 15
Concrete (poured in place)	m^3	$ 400 - 500
Trash Rack (metal)	m^2	$ 100
Inverted Elbow Pipe	each	$ 300
Outlet Gate Valves (300 mm / 600 mm)	each	$ 1,200 / $ 4,800
Outlet Sluice Gates (300 mm / 600 mm / 900 mm)	each	$ 5,500 / $ 8,000 / $11,500
Clear Stone (gravel, 25 mm ~ 50 mm)	m^3	$ 45
Filter Cloth	m^2	$ 3
Filter Material (sand)	m^3	$ 30
Vegetative Plantings		
Seed and Topsoil	m^2	$ 2.5
Grass Sod and Topsoil	m^2	$ 4.5
Emergent and Submergent Vegetation	m^2	$ 12
Shoreline Fringe and Flood Fringe Vegetation	m^2	$ 12
Upland Vegetation	m^2	$ 5
Trees (wooded filter strips)	m^2	$ 25

Note: Cost estimates are based on construction costs in Greater Toronto Area (1997-1998).

7.3 SWMP Operation and Maintenance Costs

Operation and maintenance is required to ensure effective operation, longevity and aesthetic functioning of the SWMP and may include: sediment removal, trash removal, maintenance of vegetation and inspection of the inlet(s) and outlet(s).

Different types of SWMPs require different types of maintenance activities, and costs vary with type and size. Table 6.1 in Chapter 6 lists commonly required maintenance activities for various SWMPs.

Based on monitoring data to date, infiltration SWMPs have the shortest longevity of any practice (see Chapters 4 and 6). Pre-treatment practices cannot remove all suspended solids from the stormwater. Therefore, all infiltration SWMPs eventually become clogged. Once this occurs, the entire infiltration SWMP will need to be re-constructed. In comparing maintenance costs, the re-construction cost of the entire infiltration SWMP should be used based on estimated longevity.

Estimates of the longevity of infiltration SWMPs are based on professional opinion. Equation 7.1 and Table 7.4 may be used as guidance for estimating longevity (based on monitoring results in literature and the native soil permeability). Recognizing the subjectiveness of Equation 7.1, there needs to be flexibility in assessing the lifespan of infiltration SWMPs based on site-specific information.

$$L = (P \times T)^{0.4}$$ **Equation 7.1: Longevity of Infiltration SWMPs**

where L = longevity (years)
P = permeability (mm/h)
T = longevity factor from Table 7.4 (years)

Table 7.4: Estimated Infiltration SWMP Longevity*

Infiltration SWMP	Longevity Factor
Soakaway Pit	60
Infiltration Basin	15
Infiltration Trench	25

** The values in Table 7.4 assume that adequate pre-treatment is provided upstream of the infiltration SWMPs. Without adequate pre-treatment, the expected useful life of an infiltration SWMP is considerably shorter than that given in Table 7.4 (approximately 5 years).*

This method of estimating infiltration lifespan assumes that water table and bedrock site conditions are suitable for infiltration. These conditions must be confirmed since they will also have considerable impact on the lifespan/operation of infiltration SWMPs.

Table 7.5 provides a list of unit prices for the operation and maintenance activities listed in Table 6.1. Unit prices do not include transportation and equipment costs to perform the maintenance (e.g., backhoe). It was assumed that the owner of the stormwater management works would be the local municipality, and that it would have the required equipment as part of its works department. The unit prices in Table 7.5 include labour and represent typical maintenance conditions (e.g., dewatered forebay for sediment removal). Other maintenance activities such as dredging should be costed on a site-specific basis.

Table 7.5 provides planning estimates for long-term SWMP costs. Site-specific maintenance and operation costs should be calculated wherever possible. It also indicates that the frequency of sediment removal depends on the SWMP type and design storage volume. The required frequency of sediment removal can be estimated using information provided in Chapter 6.

Table 7.5: Unit Costs for Operations and Maintenance

Type of Maintenance	Maintenance Interval (yrs)	Unit	Price
Litter Removal	1	ha	$ 2,000
Grass Cutting	***	ha	$ 250
Weed Control	1	ha	$ 2,500
Vegetation Maintenance (Aquatic/Shoreline Fringe)	5	ha	$ 3,500
Vegetation Maintenance (Upland/Flood Fringe)	5	ha	$ 1,000
Sediment Removal (front end loader)	*	m^3	$ 15
Sediment Removal (vacuum truck or manual)	*	m^3	$ 120
Sediment Testing (lab tests on quality)	*	each	$ 365
Sediment Disposal (off-site landfill)	*	m^3	$ 300
Sediment Disposal and Landscaping (on-site)	*	m^3	$ 5
Inspection (Inlet/Outlet, etc.)	1		$ 100
Pervious Pipe cleanout (flushing)	5	m	$ 1
Pervious Pipe cleanout (Radial Washing)	5	m	$ 2
Seasonal Operation of Infiltration By-pass	0.5	**	$ 100
Infiltration Basin Floor Tilling and Re-vegetation	2	ha	$ 2,800

* *Frequency of sediment removal depends on SWMP type and volume.*

** *Dependent of infiltration facility (based on centralized facility). Seasonal operation of a system with many inlets (i.e., pervious pipe system) would be more expensive.*

*** *No grass cutting or minimal frequency of grass cutting (once or twice per year).*

7.4 Engineering and Contingency Costs

Engineering and contingency costs vary significantly from site to site and project to project. For planning purposes, costs can be estimated based on experience with other consulting projects.

7.4.1 Engineering Costs

Engineering costs include the planning, design and construction of all stormwater management works/measures. For planning purposes, engineering costs are based on the total capital cost of the stormwater works. As a general estimate, the engineering cost of a SWMP can be estimated as 10% of the total construction cost for that SWMP.

7.4.2 Contingency Costs

Contingency costs represent the unforeseen costs that may occur during the construction of a SWMP. These may include additional construction costs (i.e., bedrock excavation, dewatering, etc.), additional material costs and design alterations. Actual contingency costs will vary significantly from project to project, but is generally estimated to be about 15% of the total projected construction cost for a project.

The estimated contingency costs for stormwater management works that require maintenance (i.e., facilities that are designed for either water quality enhancement or erosion control using extended detention with a drawdown time ≥ 12 hours) should be estimated as 15% of the total of the construction cost and the present value of operations and maintenance costs.

7.5 SWMP Overall Cost Calculation

The total capital cost and maintenance cost of a SWMP can be estimated using Tables 7.1 through 7.5. The following steps can be used to calculate and compare present value capital and maintenance costs between various SWMP solutions.

Step 1: Review Table 7.1 to ensure that all capital cost items for a selected SWMP type have been identified.

Step 2: Review Table 7.2 to identify the need for pre-treatment SWMPs based on the selected SWMP. Review Table 7.1 to identify the capital cost items required for the pre-treatment SWMP.

Step 3: According to the preliminary design of the selected SWMP type, and pre-treatment SWMP requirements, estimate the required quantities of each of the capital cost items (including any site-specific requirements not identified in Table 7.1) identified in Steps 1 and 2.

Step 4: Use Table 7.3 (unit prices) to calculate the capital cost of each item identified in Steps 1 and 2 (capital cost is the product of the unit price and the required quantity), and then calculate the total capital cost as the sum of the costs of all the required capital cost items.

Step 5: Review Table 6.1 to identify the operations and maintenance activities that are required for the selected SWMP.

Step 6: Use Table 7.5 to identify the required maintenance interval and the unit price for each of the required operation/maintenance activities.

Step 7: Estimate quantities for the required operation/maintenance activities according to the preliminary design of the selected SWMP; and then calculate the maintenance cost for each activity. The quantity of accumulated sediments and sediment removal frequency can be estimated using figures and tables in Chapter 6.

Step 8: Group the operations and maintenance costs for activities that are performed numerous times per year into an annual maintenance cost. Sum up other operation/maintenance costs that have the same frequency of occurrence (i.e., 2 year, 5 year, 10 year, etc.). The summations should include re-construction of infiltration SWMPs when necessary based on Table 7.4 and Equation 7.1.

Step 9: Calculate the present value of operations/maintenance activities for similar frequency occurrences. Equation 7.2 can be used to calculate the present value of these activities based on an annual interest rate and service life of the SWMP.

$$PV = \sum_{t}^{T} [\, OM \times (1 + r)^{-t} \,]$$

Equation 7.2: Present Value

where PV = Present Value
OM = Sum of operations/maintenance costs that are required to be performed every t years
t = Interval between maintenance activities in years (i.e., if the interval is 3 years, the summation proceeds with t = 3, then t = 6, then t = 9, etc., until t = T)
r = annual interest rate
T = the service life of the selected SWMP

Equation 7.2 is best utilized in a simple spreadsheet format. The service life and interest rate are user defined. Typical values would be a 50 year service life and 3% interest rate (interest rate should be discounted to account for inflation, i.e., 8% interest rate – 5% inflation rate = 3% interest rate).

Step 10: Add the total capital cost (plus engineering cost) to the sum of all the present values for operation/maintenance activities. The contingency cost should then be added to the resultant number to obtain the total present value of the cost of implementing the SWMP.

Steps 1-10 can be used to estimate overall costs for each SWMP being considered and can facilitate cost comparisons between SWMPs.

7.6 Land Requirements for End-of-Pipe Stormwater Management Facilities

End-of-pipe stormwater management facilities (wet pond, dry pond, wetland and infiltration basins) require the use of land which might otherwise be available for development (since SWMPs should be located on the tableland and not in the floodplain). It is important to recognize this when estimating the area of developable land and the area required for these types of SWMPs. If land costs for different SWMP solutions are known, they can be added to the overall calculation of costs (Step 10 value – Section 7.5) and compared. Land costs are extremely variable, and therefore, land cost estimates can be contentious. In order for land cost estimates to be meaningful, site-specific data must be used in the analysis.

The cost of land depends on its location and size. The area of land required by an end-of-pipe stormwater management facility depends on the design storage volume, the side slopes, and its shape. The following sections provide methods to estimate the area of land required by a SWMP. It should be stressed that in order to simplify the analysis, the following equations were derived using specific assumptions concerning the side slopes and shape of the various SWMPs. Calculation results will only result in planning level information which can be used to compare SWMP concepts. The actual size of the SWMP will depend on the existing topography, servicing options and surrounding natural features.

7.6.1 Wet Ponds and Wetlands

The following configuration was assumed to be indicative of typical design parameters for wet ponds and wetlands:

- bottom of the wet pond/wetland was assumed to be rectangular in shape;
- length-to-width ratio of 3:1;
- side slopes of 4:1 within the permanent pool; and
- side slopes of 5:1 in the extended detention portion of the pond/wetland.

Based on these assumptions the area of land required by a wet pond or wetland can be estimated as follows:

Step 1: Determine the width of the bottom of the pond or wetland (X in metres).

$$X = \frac{\sqrt{256h_p^4 - 12h_p\left(\frac{64}{3}h_p^3 - PV\right)} - 16h_p^2}{6h_p}$$

Equation 7.3: Wet Facility Bottom Width

where h_p = the average depth of the permanent pool (m)
PV = the permanent pool volume (m^3)

Step 2: Determine the depth of the extended detention in the wet pond/wetland (h_e in metres).

$$h_e = \frac{\sqrt{(X + 8h_p)^2(3X + 8h_p)^2 + 20(3X + 8h_p)EV}}{10(3X + 8h_p)} - \frac{X + 8h_p}{10}$$

Equation 7.4: Active Storage Depth

where EV = extended detention volume (m^3)

Step 3: Determine the area of land required for the wet pond or wetland (LA in m^2).

$$LA = (X + 8h_p + 10h_e)(3X + 8h_p + 10h_e)$$

Equation 7.5: Wet Facility Area Requirement

7.6.2 Dry Ponds and Infiltration Basins

The same calculation method can be used to estimate the land area required by dry ponds and infiltration basins. The following configuration was assumed to be indicative of typical design parameters for dry ponds and infiltration basins:

- bottom of the pond/basin was assumed to be rectangular in shape;
- length-to-width ratio of 3:1; and
- side slopes of 5:1 in the pond/basin.

Step 1: Determine the width of the bottom of the pond/basin (X in metres).

$$X = \frac{\sqrt{400h^4 - 12h\left(\frac{100}{3}h^3 - EV\right)} - 20h^2}{6h}$$

Equation 7.6: Dry Facility Bottom Width

where EV = the design extended detention volume (m^3)
h = the average depth of the extended detention storage (m)

Step 2: Determine the area of land required for the infiltration basin or dry pond (LA in m^2).

$$LA = (X + 10h)(3X + 10h)$$

Equation 7.7: Dry Facility Land Requirements

It should be noted that the area of land required is not linearly related to the design storage volume of the SWMP. Accordingly, extrapolation should not be used to calculate the land area in cases where different design storages are considered.

7.6.3 Acceptable Ranges of Design Parameters

The calculation of land area in Sections 7.6.1 and 7.6.2 requires the average permanent pool depth (wet facilities) and active storage depth (dry facilities) as inputs. Acceptable ranges of these design parameters are summarized in Table 7.6. Detailed discussions on the requirements of the design parameters of various end-of-pipe stormwater management facilities are provided in Chapter 4.

Table 7.6: Acceptable Ranges of Design Parameters

Design Element	Wet Pond	Dry Pond	Wetland	Infiltration Basin
Permanent Pool Depth	1 to 3 m		0.15 to 0.30 m	
Extended Detention Storage Depth	1 to 1.5 m	1 to 3 m	≤ 1 m	≤ 0.6 m

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APPENDIX A
SUBDIVISION/SITE PLANNING

A.1 Overview of Process

The term subdivision/site planning applies to subdivision planning, site planning and engineering, landscape design, architectural and building design, as well as local street design. Integrated subdivision/site planning is an effective means to ensure that parallel social, environmental, economic and functional objectives are achieved. The salient aspects of this process are described below.

- **Establish Objectives**
  Based upon an understanding of the natural features, context and the vision for future use of the site, a multi-disciplinary team should establish specific ecological, social, functional and economic objectives. This approach ensures that all objectives are defined and initiates the process of identifying parallel objectives, which is essential to achieving integrated solutions.

- **Set Targets**
  Related to each objective, identify specific performance criteria or design parameters. These 'targets' will guide the exploration of solutions and ensure that all necessary elements are addressed in the final design, including stormwater management, in terms of quality improvement and quantity control.

- **Establish Objectives Identify Techniques**
  The goal of this step in the process is to explore the range of techniques that could be employed to address each target. This should be done with an emphasis on research and innovation, rather than acceptance of standard solutions. It is at this stage in the process that the overlap of techniques, which yields integrated solutions to achieve multiple objectives, begins to become evident.

- **Explore Opportunities**
  Opportunities to achieve more than one objective through the application of single or multiple techniques should be identified. The unique attributes of the site and its context are the basis for the exploration of opportunities. The design team should collectively evaluate opportunities in order to ensure that objectives are addressed with a balanced perspective and to facilitate the thoughtful resolution of conflicts between competing objectives or contrary techniques.

- **Generate Conceptual Alternatives**
  Opportunities should be assessed to confirm suitability, practicality and compatibility with legislative requirements. Opportunities assessed and determined to be feasible are

then integrated into a comprehensive plan or plans, which illustrate a conceptual alternative for the integrated design of the site.

- **Develop the Final Plan**
  Through an interactive process of design, evaluation and refinement, the final plan is evolved from the concept plan. Individual components of the final plan should be resolved with a continued emphasis on innovation within a multi-disciplinary forum. The final plan should not only address the implementation of physical initiatives, but also the recommendation of management-based solutions.

A.2 Subdivision/Site Planning and Stormwater Management Practices

It is important to understand that subdivision/site planning is a fundamental determinant of the overall change in the hydrologic cycle for a given development. However, the significance of subdivision/site planning is not always well understood by the landowners, their consultants, local decision makers or the public. The following discussion provides an appropriate framework to understand this important aspect of the development process.

- **Watershed and subwatershed planning: environmentally responsible land use policies must be supported by environmentally responsible site design.**

  The preparation of watershed and subwatershed plans is recognized as an essential part of the land use planning process. The watershed and subwatershed planning process is integrated with the official plan preparation and review process to ensure that an ecosystem approach is adopted in making land use planning decisions.

  Watershed and subwatershed plans address the ecosystem at a regional level. At this level, land use decisions are made as generalized policies and guidelines, and environmental information is often collected and interpreted at a broad scale. While these broad scale evaluations allow the development of strategies which are not possible through site specific evaluations, it is not always possible to interpret the merits or demerits of various individual development proposals at this stage.

  The fundamental objectives of watershed and subwatershed planning can only be realized if the principles of watershed/subwatershed planning are also applied during the planning and design of individual development projects. At this point of the development process, detailed site information is available and the physical parameters of the proposed development are determined. The subdivision/site planning stage is therefore an important step in the planning process when the impact of the development proposal on the environment can be specifically assessed. The integration of land use planning and environmental planning at a regional or district level must be extended to the process of site development and design.

- **Good planning integrates the design of a site and the design of the stormwater management facilities in one process.**

  Historically, the preparation of subdivision plans, site development plans as well as building and architectural design plans has not involved early input from environmental planners, hydrogeologists, ecologists and water resources engineers. The landowners and the planners/designers prepare the plan based on the performance standards set by the municipal by-laws or guidelines (such as setback, floor space index, density, height, etc.), and the business objectives set by the landowners (such as total leasable floor area to be achieved, number of units for sale and the number of parking spaces to be provided).

  Water resources engineers, and other associated professionals, are typically employed to address stormwater management after a preliminary site plan has been prepared. This process has inevitably made the proposed stormwater management facilities 'remedial' in nature since they are designed to handle a predetermined amount of runoff and to mitigate the negative impact of the proposed development. An alternative approach is advocated. The objective of reducing the root causes of negative impact on water management should be adopted as one of the basic design criteria directing the preparation of the site plan. The important aspect of good subdivision/site planning is that it should aim at reducing or preventing adverse impacts instead of mitigating them.

- **Public perception and implementation of innovative subdivision/site planning approaches.**

  There is a perceived public attitude that many of the proposed environmentally friendly subdivision/site planning techniques such as cluster housing forms, roadside ditches and the inclusion of runoff infiltration devices within residential lots are undesirable and represent a reduction in the level of service. This perception extends to some municipalities whose development standards may constrain the use of innovative subdivision/site planning techniques. As a result, developers may hesitate to include these design alternatives in their site development plans. Nevertheless, the attitude of the public is changing as more innovative projects are delivered into the market and the public sees the value of these new design concepts. Creative stormwater management design ideas should be encouraged and adopted as part of the design during the subdivision/site planning stage of the development process.

- **The most environmentally sound design is generally the most economical.**

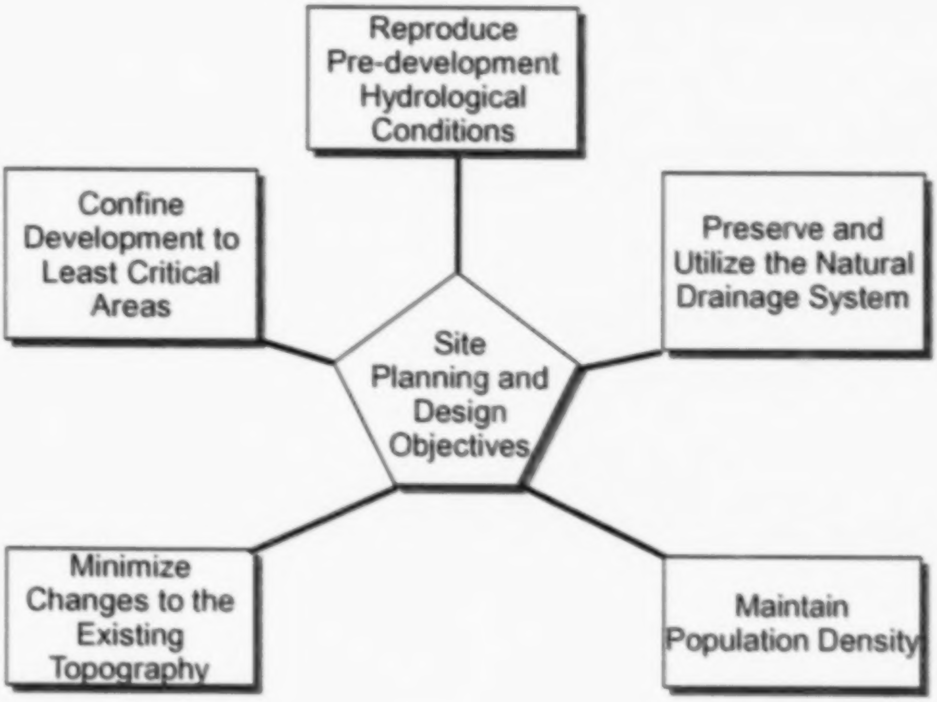
  Subdivision/site planning generally reduces the cost of the development due to:

  - lower grading requirements/costs;
  - lower tree clearing costs;
  - lower servicing costs (swales instead of storm sewers);
  - lots with mature trees are more saleable/valuable;
  - lots that back on to greenbelts are more saleable/valuable;
  - tourism dollars in areas with sports fishery; and
  - lower end of system clean up costs (i.e., dredging, etc.).

A.3 Subdivision/Site Planning and Design Objectives

There are many excellent references, such as "Protecting Water Quality in Urban Areas – Best Management Practices for Minnesota" (Minnesota Pollution Control Agency, 1989) which illustrate the value of subdivision/site planning. These references were reviewed in the formalization of the objectives shown in Figure A.1. Design decisions made during the subdivision/site planning stage of a project should be assessed against these objectives.

Figure A.1: Subdivision/Site Planning and Design Objectives


Reproduce Pre-development Hydrological Conditions
Confine Development to Least Critical Areas
Preserve and Utilize the Natural Drainage System
Site Planning and Design Objectives
Minimize Changes to the Existing Topography
Maintain Population Density

A.4 Subdivision/Site Planning Methodology

To assist site designers, the objectives have been translated into a subdivision/site planning methodology which may be used to prepare a development layout. The process may be summarized as follows:

1. **Agency Consultation** – identify existing resource mapping/data and natural resource concerns.

2. **Resource mapping** – identify significant natural functional areas for protection.
3. **Designation of development area** – determine the areas for development based on the resource mapping information.
4. **Evaluate stormwater management requirements** based on the preliminary site plans. Indicate locations and land area to be formalized in the site plan for the purposes of stormwater management.
5. **Adoption of environmentally responsible site planning and design criteria** – apply a set of environmentally responsible design criteria to the development area during the preparation of the site plan options.
6. **Finalization of the subdivision/site layout** – examine the various site plan options based on the criteria and select the option that best meets the site planning and design objectives.

A.4.1 Agency Consultation

The regulatory agencies (Local Municipality, Ministry of Natural Resources, Ministry of the Environment, Conservation Authority) should be contacted for information on existing areas which are deemed to be environmentally significant.

A.4.2 Resource Mapping

Resource mapping is required to ensure that significant natural resources are maintained or enhanced. On an appropriate scale ($\leq$ 1:2000) map of the proposed development site an outline of the following resources should be clearly delineated:

- ESA/ANSI areas;
- watercourses, lakes and other water bodies;
- wetlands;
- significant vegetation/woodlots;
- wildlife corridors;
- high recharge potential areas;
- regulatory floodlines and/or fill lines;
- stream and valley corridors;
- bank instability and erosion setbacks; and
- steep sloped areas.

Much of the information required for resource mapping may have been delineated (usually at a larger scale) in the watershed or subwatershed plan (if it has been completed). Reference should be made to these plans as part of the site investigations.

ESA/ANSI Areas
The Ministry of Natural Resources and the Conservation Authority should be contacted for mapping which indicates Environmentally Sensitive Areas (ESA) and Areas of Natural and

Scientific Interest (ANSI). Municipalities should also be contacted for mapping related to any Locally Significant Areas (LSA). These areas should be transferred to the site mapping and be clearly shown on development submissions.

Watercourses, Lakes and other Water Bodies

Watercourses, lakes, and other water bodies should be denoted on the resource mapping. Ontario Base Mapping (1:2000, 1:10000), where available, is a useful source of information which will indicate surface water resources. Larger scale topographical mapping will also indicate most surface water resources. In some cases, however, not all surface water resources may be delineated to preserve clarity (i.e., in areas with high topographical relief – many contours). In all instances, a site visit should be undertaken to confirm the surface water resources in the vicinity of the proposed development.

Wetlands

Wetlands should be shown on the resource mapping and Provincially Significant Wetlands should be identified. An environmental impact study (EIS) will generally be required if development encroaches within 120 m of a Provincially Significant wetland boundary, and it may be required for other wetlands as well. This study will assess the potential impacts of development on the wetland and recommend an appropriate buffer width and/or other mitigative measures.

Areas of Significant Vegetation

A terrestrial biologist should walk the site to identify the areas of the site with significant vegetation. Significant vegetation includes provincially significant, regionally significant, and locally significant species. An area can also be deemed significant, in terms of its vegetation, if it provides a corridor or refuge area for wildlife, a food source for terrestrial/aquatic species, a significant hydrological function, and/or a buffering capacity to mitigate the effects of urban development on the stream and valley corridor system.

In some cases, information on the vegetation of a site can be obtained from the Conservation Authority, Ministry of Natural Resources, and/or local naturalist groups. However, where mapping/information is dated, a site walk/inventory should be done. Not only may site conditions have changed, but also, values with respect to the importance of vegetation have evolved dramatically and may influence the mapping/information collected. The limit of development should be the drip line of the vegetation. No earthworks should be permitted within 3 to 5 metres of the vegetation drip line to protect root systems.

Wildlife Corridors

The significance of wildlife corridors is best addressed in Watershed or Subwatershed Plans. These plans should be reviewed if they exist. If a watershed plan and/or subwatershed plan has not been completed, the Ministry of Natural Resources should be consulted for input. A site walk by a terrestrial biologist should be undertaken to confirm the recommendations of the watershed/subwatershed plan and the information provided by the Ministry of Natural Resources. Information from the site walk should be compared to the greenspace areas in the surrounding geographic area to determine if a wildlife corridor exists on the site. Significant wildlife corridors should be drawn on the resource map.

Recharge Areas

Boreholes and test pits are required to determine the groundwater recharge potential for the site. This investigation must be undertaken by a qualified soils consultant or geotechnical engineer. Information which needs to be collected includes soil types, soil depths, the depth to the water table, the degree of soil compaction, soil percolation rates, the estimated high seasonal water table depth, the depth to bedrock, and soil particle size distributions.

Percolation rates measured in the field may be used as an indicator of the potential for groundwater recharge. Areas with a percolation rate greater than 50 mm/h should be identified as important recharge areas (i.e., any development must ensure that recharge is maintained), and areas with a percolation rate greater than 100 mm/h should be identified as critical recharge areas (i.e., areas that may be non-developable or require significant investigation in support of development), given that the depth to bedrock and depth to the water table are greater than 3 m below the ground surface.

It is important to identify other hydrogeologically sensitive areas, such as locations where aquifers may be susceptible to contamination due to their proximity to the surface and the nature of surficial deposits.

Regulatory Floodline and/or Fill Line

The regulatory floodline and/or fill line should be shown on the resource map if the proposed development is adjacent to a watercourse. If a floodline or fill line has not been delineated, and is not required to be delineated (i.e., upstream drainage area is small (< 125 ha) and the Conservation Authority is not concerned with flooding) it does not need to be shown on the map of the site. In cases where the flood or fill line is not shown, the watercourse should still be shown as it may serve an important ecological function.

Stream and Valley Corridors

The area required to protect stream and valley corridors is best decided at the subwatershed plan level. The stream and valley corridor area should be shown on the resource map.

Bank Instability and Erosion Areas

Areas susceptible to bank instability and erosion should be identified on the resource map. These areas will typically be within the stream and valley corridors. Tables C.1 and C.2 provide guidance on identifying areas susceptible to erosion.

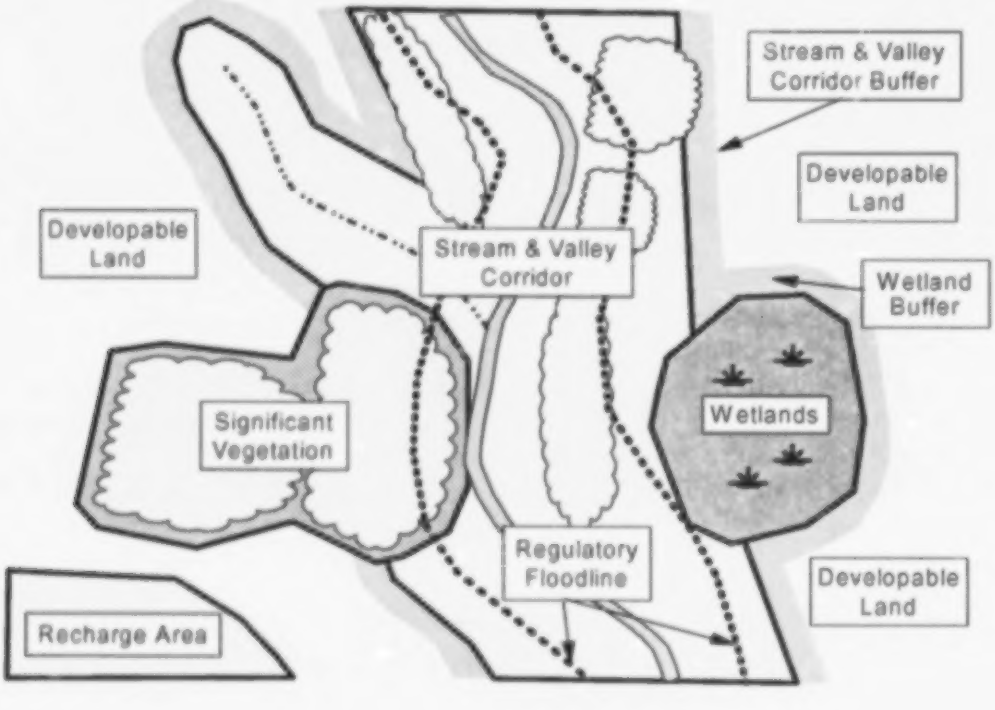
Steep Sloped Areas

Areas with a slope of greater than 20% should be identified on the resource map. These areas may be difficult to develop (i.e., result in significant alteration to the natural topography) and should be noted as constraint areas.

A.4.3 Designation of Development Area

The resource mapping information should be compiled into overlays of information sheets and maps for easy cross referencing. These overlays will illustrate the inter-relationship between the different elements of the ecosystem. At this stage, the planner/designer should determine where development should occur within the site to minimize impacts on the environment. Figure A.2 illustrates the concept of resource mapping to determine developable land. Once the area of developable land has been identified, a development layout should be prepared based on a set of environmentally responsible subdivision/site planning and design criteria.

Figure A.2: Resource Mapping


Stream & Valley Corridor Buffer
Developable Land
Developable Land
Stream & Valley Corridor
Wetland Buffer
Wetlands
Significant Vegetation
Regulatory Floodline
Developable Land
Recharge Area

A.4.4 Reserve Appropriate Areas for Stormwater Management

Subdivision/site planning must reflect the need for stormwater management. This requires interaction between planners/designers and stormwater management professionals to ensure that there is adequate land area in appropriate locations designated for the purpose of stormwater management. The requirements for stormwater management will depend on the water management criteria which have been established for the site, the stormwater management measures that are contemplated, and the actual site planning that is proposed. The full range of stormwater management measures (lot level, conveyance, end-of-pipe) should be contemplated. At this stage, preliminary design and siting of stormwater management controls would be appropriate.

Urban stormwater management practices should be located outside of the floodplain wherever possible. In some site specific instances SWMPs may be allowed in the floodplain if there is sufficient technical or economic justification and given that they meet certain requirements:

- The cumulative effects resulting from changes in floodplain storage, and balancing cut and fill, do not adversely impact existing or future development;

- Effects on corridor requirements and functional valleyland values must be assessed. SWMPs would not be allowed in the floodplain if detrimental impacts could occur to the valleyland values or corridor processes;

- The SWMPs must not affect the fluvial processes in the floodplain; and

- The outlet invert elevation from any SWMP should be higher than the 2 year floodline and the overflow elevation must be above the 25 year floodline.

In most cases, online facilities (those located within a watercourse) are discouraged because of concerns for wildlife movement, fish passage and disruption of energy inputs. Online stormwater quantity facilities may be acceptable if designed such that the bankfull flows, and hence fish movement, are not impeded/obstructed, and provided that the foregoing requirements are met. Online quality ponds can only be approved if issues of aquatic habitat can be resolved. An online facility could only be proposed in the context of a subwatershed plan.

The location of end-of-pipe stormwater management facilities is a contentious issue since the use of tableland reduces the overall developable area. In an effort to minimize the loss of developable land municipalities can consider the use of parkland dedication for SWMPs which offer passive recreational opportunities and follow the municipality's greenland strategies (parkland objectives) wherever possible.

A.4.5 Adoption of Environmentally Responsible Subdivision/Site Planning and Design Criteria

The following general planning and design criteria are recommended:

- preserve existing topography and natural features;
- protect surface water and groundwater resources (stormwater management);
- adopt compact development forms;
- adopt alternative site development standards; and
- re-create natural habitats within the development areas.

These criteria, and techniques which can be used to accomplish them, are discussed in the following sections.

Preserve Existing Topography and Natural Features

In order to preserve the existing topography and natural drainage system, buildings and roads should be located along high points and on flat slopes (Figure A.3). Natural drainage swales should be used to convey runoff from the development to the receiving waters (Figure A.4). This approach will reduce the area disturbed by cutting and filling along the slope and minimize the amount of surface area susceptible to erosion.

Figure A.3: Preservation of Existing Topography

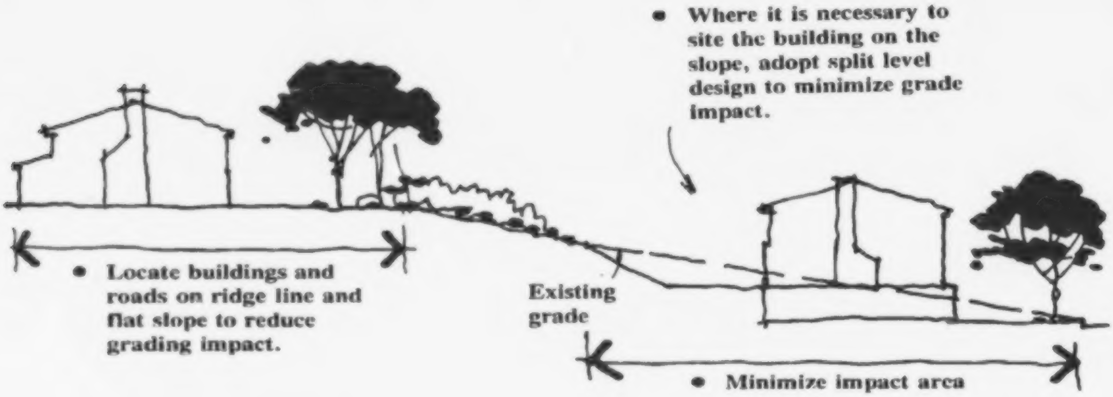
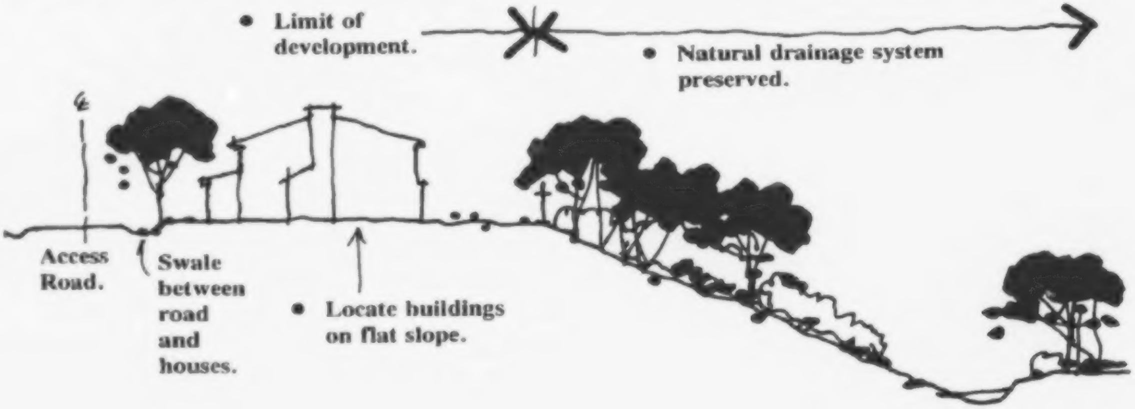

• Where it is necessary to site the building on the slope, adopt split level design to minimize grade impact.
• Locate buildings and roads on ridge line and flat slope to reduce grading impact.
Existing grade
• Minimize impact area

Figure A.4: Preservation and Utilization of the Natural Drainage System


• Limit of development.
• Natural drainage system preserved.
Access Road.
Swale between road and houses.
• Locate buildings on flat slope.

The application of this criterion must be made with consideration for the visual impact of locating buildings on and along the ridgelines of the landscape. To avoid the visual intrusion of buildings along attractive natural ridgelines and the disruption of existing prominent landforms, it may be necessary to site the buildings and the access roads along the contouring slopes.

Protect Surface Water and Groundwater Resources
The concerns with respect to surface and groundwater resources must be identified and the level of control required to address these concerns must be defined. The site plan should adopt a combination of lot level, conveyance and end-of pipe stormwater management approaches that will mitigate the effects of urbanization on surface and groundwater resources. The constraints and opportunities presented by the physical site conditions (e.g., site hydrology and soils) must be considered in the selection of stormwater management controls.

Adopt Compact Development Forms
Adoption of compact housing forms such as cluster single dwellings, medium density townhouses and low-rise apartments, and high-rise apartments can compensate for restrictions in the area of developable land due to environmental features. A certain level of development density may be achieved while reducing the extent of disturbance to the site and the amount of site works required. Figure A.5 illustrates the concept of maintaining density with single detached cluster housing while reducing the overall development area. The feasibility of single detached cluster housing is dependent on the use of alternative development standards.

The Ministry of Municipal Affairs and Housing promotes compact, higher density housing forms. Compact, higher density housing forms are shown in Figures A.5 and A.6, and may include:

- cluster single lots with reduced lot frontages and alternative road/grading standards;
- higher density forms such as duplex and semi-detached;
- condominium singles;
- medium density housing forms such as townhouses, fourplex and low-rise apartments; and
- high density housing such as high-rise apartments.

Adopt Alternative Site Development Standards
Many of the compact development forms recommended above can only be implemented with flexible site design standards (building setbacks, grading requirements, minimum street gradient and turning radius, width of internal streets, locations of site services, provision of street boulevard areas).

Alternative development standards are generally allowed in non-freehold development projects (i.e., projects in which the services (roads, stormwater management facilities, etc.) are not municipally maintained – such as condominiums). Any public right-of-ways, public areas, and freehold residential lots, however, have to comply with the normal municipal planning and engineering (grading, servicing) standards. Public streets are designed to have a wide right-of-way and gentle gradients. These standards may limit the implementation of alternative housing forms to non-freehold developments. The adoption of alternative cluster single lots for the typical freehold development, for example, will be less effective if alternative development standards are not utilized.

Alternative development standards complement reduced lot frontages and depths to reduce the overall development footprint. "Making Choices: Alternative Development Standards Guideline" (Ministry of Municipal Affairs and Housing, 1995) reviews municipal standards and recommends alternative standards to reduce development costs, promote compact urban form, and mitigate environmental impacts.

Figure A.5: Cluster Single Detached Dwellings

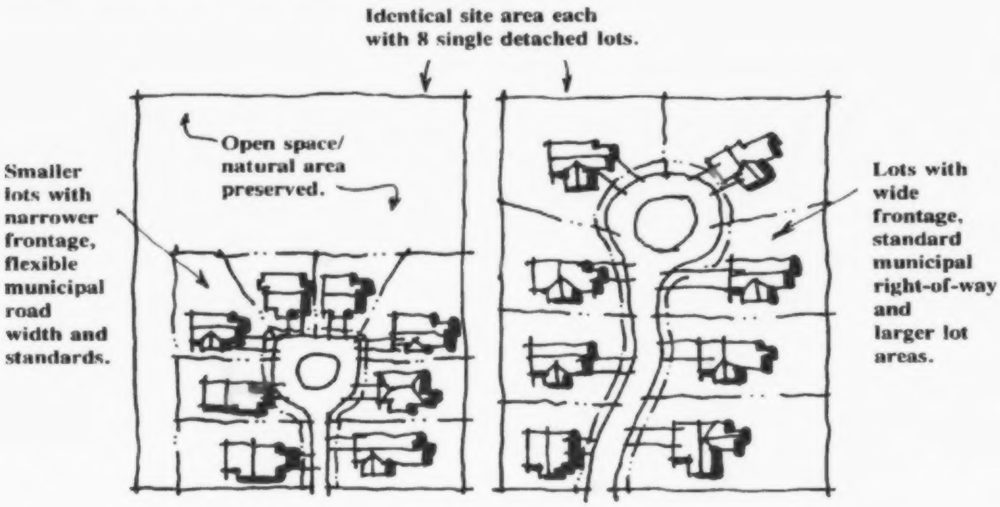
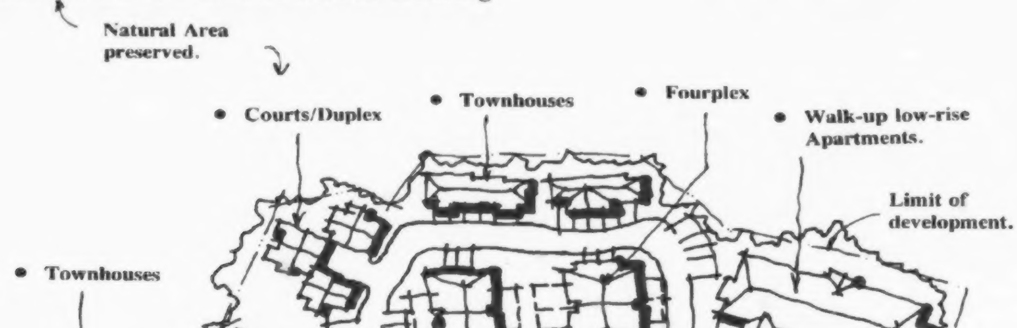

Identical site area each with 8 single detached lots.
Open space/ natural area preserved.
Smaller lots with narrower frontage, flexible municipal road width and standards.
Lots with wide frontage, standard municipal right-of-way and larger lot areas.

Figure A.6: Other Forms of Cluster Housing


Natural Area preserved.
• Courts/Duplex
• Townhouses
• Fourplex
• Walk-up low-rise Apartments.
Limit of development.
• Townhouses
Natural area preserved.
Limit of development.

Some alternative engineering standards which help to reduce the overall footprint of development include:

- reduced road widths on local roads

  Reducing the road width to 6 m on local roads allows for two way traffic without street parking or one way traffic with parking. This reduces the overall pavement area, and hence costs, for the subdivision. The reduction in the pavement area will minimize the amount of land to be disturbed and grading works. It will also provide more flexibility for the planner/designer to align the proposed road along existing contours and integrate it into the existing landform.

- reduced cul-de-sac turning radius

  A reduction in pavement and overall land consumption can be achieved if the cul-de-sac turning radius is reduced from 14 to 11 metres.

Other alternative engineering standards which minimize environmental degradation and changes to the natural function of the land are shown in Figure A.7 and include:

- a wider range in allowable lot grading

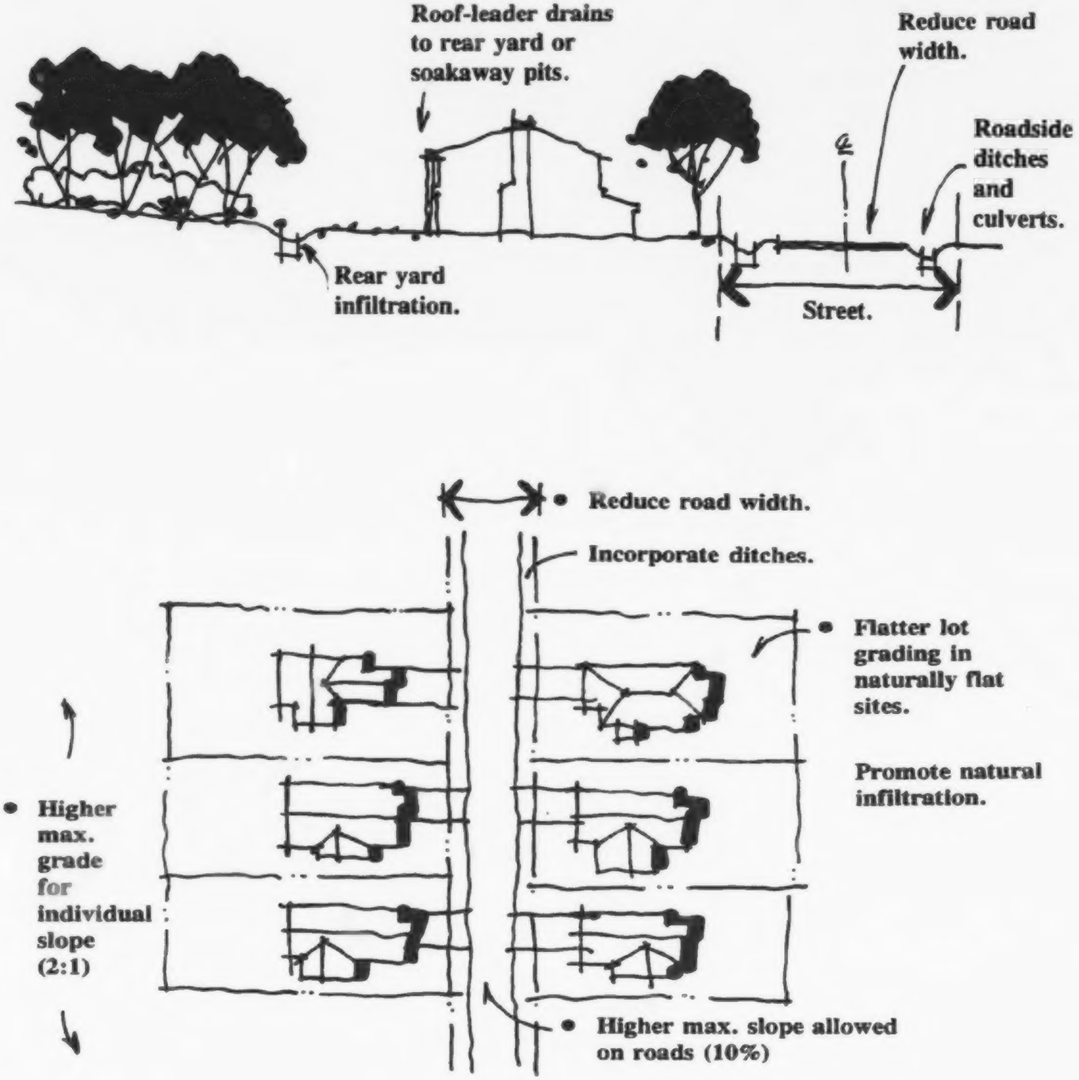
  A reduction in the minimum allowable lot grade promotes natural infiltration and creates greater depression storage. Due to the problems of physically being able to grade below 2%, there should be an elevated apron around buildings (within 2 to 4 metres) to ensure that water does not drain towards the building foundation.

  Flatter lot grading should be promoted in naturally flat areas but radical changes to the existing topography should not be made. Municipal grading standards may also need to be modified for development within areas of varying topography to permit steeper lot grading. This flexibility will assist the designer to site the buildings along the slope and fit the built form into the terrain with minimum disturbance to the existing topography.

- higher maximum allowable slopes on roads (10% instead of 6%) and individual lots (2:1 instead of 3:1)

  The increase in range of maximum allowable slopes allows planners/engineers greater flexibility in designing developments within the existing topography. Economic and environmental benefits accrue from reduced grading requirements, although there may be some drawbacks such as greater requirements for sanding/salting these roads during the winter and increased erosion potential in roadside ditches. On the other hand, narrower road surfaces will also mean reduced amounts of road salt/sand and lower construction costs. These issues are best addressed from a holistic perspective recognizing the environment, the economy, and the functionality of the subdivision/site design.

Figure 1.7: Alternative Development Standards


Roof-leader drains to rear yard or soakaway pits.
Reduce road width.
Roadside ditches and culverts.
Rear yard infiltration.
Street.
• Reduce road width.
Incorporate ditches.
• Flatter lot grading in naturally flat sites.
Promote natural infiltration.
• Higher max. grade for individual slope (2:1)
• Higher max. slope allowed on roads (10%)

- discharge of roof leaders to soakaway pits or rear yards for natural infiltration/evaporation

  Water that is discharged from roof leaders is relatively clean water. The only potential contamination of this water is by atmospheric deposition and roofing materials. Options that promote the infiltration of this water into the surrounding native soil material are promoted since they reduce peak flows and enhance groundwater/baseflow recharge. Roof leaders discharge to the surface should be minimum standard practice even in areas where there are physical constrains on infiltration.

- servicing via enhanced grassed swales and culverts instead of storm sewers

  The use of grassed swales (commonly referred to as ditch and culvert servicing) is viable for lots which will accommodate swale lengths ≥ the culvert length underneath the driveway (not just the driveway pavement width). The swale length should also be ≥ 5 m for aesthetic and maintenance purposes. This is generally achievable for small lots (9 m) with single driveways or larger lots (15 m) with double driveways. Grassed swales provide numerous benefits (water quality enhancement, reduction of water quantity peak flows and volumes, easier snow removal, storage for snow removal) and are recommended for implementation wherever feasible.

- foundation drains to soakaway pits or sump pumped to the rear yards for natural infiltration

  Foundation drainage is relatively clean water having been filtered by the backfill surrounding the foundation. Options that promote the infiltration of this water into the surrounding native soil material reduce peak flows and enhance groundwater recharge. In areas where infiltration is not appropriate (i.e., percolation rate < 15 mm/h), a separate foundation drain should be considered to reduce the volume of water being treated by any end-of-pipe stormwater management facility.

- increase rear lot overland drainage

  A greater tolerance for designs that allow overland drainage across lots is preferred from an environmental standpoint since they provide greater opportunities for reducing peak flows and stormwater volumes. Overland drainage also provides opportunities for water quality improvement through settling, adsorption, filtration, and infiltration.

  Opportunities to increase rear lot overland drainage include:

  - allowing lots backing on to one another to drain through each other; and
  - increasing the allowable length of rear yard swales and contributing drainage area.

- increase the allowable vertical sag at intersections (K of 4 instead of 10)

  An increase in the allowable elevation differences for intersection approaches will allow a development to be designed with less changes to the existing topography. This alternative standard is promoted for stop intersections, but may not be applicable for through-type intersections due to increased traffic safety concerns.

Re-create Natural Habitats within the Development Areas
Within the designated development areas, and as part of the overall subdivision/site planning concept, opportunities to recreate natural habitats should be identified. Opportunities could include selected areas within public parks, roadside revegetation with native woodland species, naturalization of any disturbed slopes, and assisted natural regeneration along existing or new watercourses.

A.4.6 Finalization of the Subdivision/Site Layout

Different design options which meet the adopted subdivision/site planning criteria will have been generated. To select a preferred subdivision/site layout, the planners/designers should evaluate the options against the objectives outlined in Section A.3. The subdivision/site layout which best satisfies these objectives should be endorsed as the appropriate development strategy.

APPENDIX B
PROPOSED PROTOCOL FOR DETAILED DESIGN APPROACH

The objective of this Appendix is to provide a checklist for the Detailed Design Approach. The checklist may be modified to fit a specific project or site as required.

STEP 1: Project Goals and Objectives, Channel Characterization and Study Scope

This STEP is designed to provide a framework for further investigations by establishing the project goals and objectives, providing a preliminary characterization of the channel system and possible disturbances, and defining the spatial scope of the investigation.

DATA COLLECTION

1. Collection and Review of Existing Documentation
    a) Land use and topographic mapping, aerial photography
        i) historic
        ii) existing
        iii) future
    b) Infrastructure mapping
    c) Background reports, surficial geology (physiographic) mapping
    d) Hydrometeorological data
    e) Regional flow-geomorphic data
    f) Historic channel surveys
        i) engineering drawings (bridge crossings, channelization works, pipeline crossings, etc.)
        ii) geomorphic-sedimentologic surveys
        iii) geotechnical studies (soils or borehole data)
2. Desktop Analyses
    a) Longitudinal channel profile
    b) Estimated bankfull flow
    c) Anticipated channel form
3. Synoptic Field Survey
    a) Site Reconnaissance and completion of a Rapid Geomorphic Assessment (RGA)
    b) Classification of Stream Type

ANALYSIS

1. Determine Total Basin Imperviousness (TIMP)
2. Assess past changes in sediment-flow regime
3. Determine tributary area
4. Re-construct land use and channel works history
5. Preliminary Mapping of 'like' reaches

6. Compare historic channel form with current form
7. Assess channel stability and probable mode of alteration
8. Assess the significance of prior disturbances on channel form
9. Determine if the channel is currently in a state of adjustment
10. Identify constraints and opportunities for Stormwater Management (SWM) measures

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STEP 2: Identification of Causative Factors

If a prior disturbance has had a significant impact on channel form and the channel is in a state of adjustment, then undertake the following analysis. Otherwise proceed to Step 3.

DATA SOURCES

1. Existing documentation (STEP 1)
2. Empirical Relations
   a) Channel Enlargement Curve
   b) Mesoscale Channel Form Relaxation Curve

ANALYSIS

1. Identify the probable cause and magnitude of the disturbance(s)
2. Select a methodology for assessment of the impact of the disturbance(s) based on (1.) above
3. If a natural phenomenon, assess whether the disturbance is endemic to the channel system or an external event
4. If the disturbance is anthropogenic in origin determine the timing and magnitude of the disturbance and the likely alteration in the flow-sediment regime. For example, if the impact is due to urbanization:
   (a) Determine the fraction of the tributary area for which land use alteration has occurred for 5 to 6 time periods (10 years for each period) beginning with the current year and moving backwards in time
   (b) Determine the TIMP for each period
   (c) Determine the area weighted average age of development (t_i) for each period
   (d) Estimate the relaxation time
   (e) Approximate the degree of completion of the adjustment process from the Relaxation Curve
   (f) Estimate the ultimate channel enlargement ratio under existing land use conditions and drainage practices from the Channel Enlargement Curve
   (g) Determine the amount of channel enlargement that is yet to occur
   (h) Determine the significance of other factors, e.g., knickpoints, sediment waves, hydraulic controls, channel works, localized perturbations in the flow regime, etc.

STEP 3: Reconstruct the Historic (Pre-Disturbance) Channel Form

The previous assessment of historic channel form represented a preliminary estimate of channel hydraulic geometry. This STEP involves a more rigorous definition of the historic channel form if deemed necessary. Otherwise proceed to STEP 4.

DATA SOURCES

1. Existing documentation (STEP 1)
2. Personal accounts
3. Empirical Relations (STEP 2)
4. Paleo-fluvial techniques
5. Field survey data (STEP 5)

ANALYSIS

1. Re-construct the pre-disturbance channel form from historic surveys and/or paleo-fluvial techniques
2. If the historic surveys were taken subsequent to the de-stabilization of the channel use hindecasting techniques, such as the Relaxation Curve, to estimate the pre-disturbance channel form
3. Confirm the hindecaste estimation of the pre-disturbance form using a regional data base (if available), geomorphic indicators (see STEP 5), personal accounts, oblique and aerial photographs, historic mapping, and/or paleo-fluvial techniques
4. Estimate the bankfull hydraulic geometry parameters

STEP 4: Assess the Impact of Future Disturbances Using Empirical Relations

Assuming that the development project were to proceed without the implementation of SWM control measures determine the probable impact on channel morphology.

DATA SOURCES

1. Existing documentation (STEP 1)
2. Empirical Relations (STEP 2)

ANALYSIS

1. If it has been determined from the pervious STEPS that the channel is evolving toward a new equilibrium position in response to a past disturbance, then this alteration in form must be accounted for in this STEP
2. Assess the impact of future land use change
    a) determine the t_i under future land use conditions
    b) determine the total directly connected impervious area under future land use conditions
    c) assess the impact of proposed SWM measures for erosion control
3. Determine the ultimate Enlargement Ratio

4. Assess the impact of other contributing Factors (assumed to be secondary to the change in flow regime associated with urban development)
5. Determine the increase in Enlargement Ratio between existing and future land use conditions
6. Identify constraints and opportunities if different from STEP 2

STEP 5: Existing Channel Dynamics

The preceding analysis have relied primarily on existing data sources, with the possible exception of the paleo-fluvial investigations. The remaining STEPS are based on the collection of field data characterizing the current channel form.

DATA SOURCES

1. Field Survey
    a) Geodetic survey of channel longitudinal profile (along the channel thalweg). A fixed longitudinal spacing for measurement of the bed profile can be adopted if the selected interval is approximately 1/5 the length of the shorter of the pool or riffle features. If a fixed interval sampling protocol is selected, measurements should also be recorded at all major break of slope points.
    b) Geodetic survey of the channel cross-section:
        i) select a representative number sites for detailed sections
        ii) select a number of sites for less detailed study
    c) For each of the detailed sections:
        i) map bank stratigraphy
        ii) characterize the bank materials
        iii) map root zone depth
        iv) determine root density
        v) characterize the riparian vegetation
        vi) complete a pebble count survey
        vii) map bankfull stage indicators
        viii) prepare photographic documentation
        ix) sketch bank profile noting location of bankfull indicators, soil strata, terraces, root zone depth, etc.
        x) sketch channel plan form geometry up and downstream of the survey section
2. Regional Data Base (if available)

ANALYSIS

1. Determine channel hydraulic geometry relations
2. Determine sediment mass curves
3. Develop shear stress vs. depth curves
4. Develop stream power relations
5. Estimate critical shear stress values for selected boundary stations

6. Plot the longitudinal profile
7. Plot the cross-sections
8. Determine hydraulic parameters such as Manning's 'n' value, water surface slope, flow rate versus depth, etc.

STEP 6: Observed Channel Response

If a significant prior disturbance has occurred, then the actual response of the channel to the disturbance must be estimated and its impact on the proposed development project assessed. Otherwise proceed to STEP 7.

DATA SOURCES

1. Field Survey (STEP 5)
2. Historic channel form (STEP 3)
3. Pre-disturbance channel form (STEP 3)
4. Empirical relations (STEP 1)

ANALYSIS

1. Determine actual Channel Enlargement Ratio using the current channel form as measured in STEP 5 and the estimated pre-disturbance form as determined in STEP 3
2. Plot the actual Channel Enlargement Ratio on the Channel Enlargement Curve to validate the estimate of ultimate channel form completed in STEP 3
3. Determine actual channel evolutionary state using the Relaxation Curve
4. Identify the mode of channel enlargement and the probable, ultimate channel plan and cross-sectional form

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STEP 7: The Need For Mitigation and the Development of Channel Remediation Strategies

Based on STEPS 4 and 6 assess the:

(a) need for mitigation of the channel due to past disturbances and the probable impact from the proposed development project
(b) develop channel restoration alternatives (if required)

DATA SOURCES

1. Dimension of the ultimate channel form (STEP 6)
2. Goals and objectives (STEP 1)

ANALYSIS

1. Determine if the ultimate channel form and its function meet the project goals and objectives
2. Based on (1.) above assess the need for and feasibility of remediation

3. Identify constraints and possible remediation strategies
4. Develop SWM design targets

STEP 8: Watershed Management Strategies

Develop a SWM program that addresses the predicted impact on channel form and function relative to project goals and objectives using the design criteria developed in STEP 7.

DATA SOURCES

1. All previous STEPS
2. Hydrologic-hydraulic and sediment transport models

ANALYSIS

1. Identify SWM alternatives (each alternative is comprised of a suite of management practices)
2. Develop a decision support algorithm for use in the evaluation of the SWM alternatives
3. Evaluate the SWM alternatives and select a preferred approach
4. Undertake the preliminary design and costing of the preferred approach
   (a) locate the required SWM facilities
   (b) develop the appropriate implementation programs
   (c) design the end-of-pipe facilities by establishing the:
      - contribution of lot level and conveyance controls
      - the active storage volume in the end-of-pipe facility
      - the rating curve for the pond outlet structure (Appendix D)

STEP 9: Selection of the Preferred Channel Restoration Strategy

Once the SWM program has been established, the final assessment of the channel restoration options may be completed resulting in the selection of a preferred restoration program. If channel restoration is not required, proceed to STEP 10.

DATA SOURCES

1. Dimension of the ultimate channel form (STEP 6)
2. Goals and objectives (STEP 1)
3. Existing data sources (previous STEPS)
4. Constraints and opportunities mapping (STEP 7)

ANALYSIS

1. Translate generic design alternatives into site specific remediation options
2. Develop cost estimates
3. Select a preferred channel restoration alternative

STEP 10: Preferred Restoration Plan

DATA SOURCES

1. Funding mechanisms
2. Cost estimate (STEP 9)
3. Land use plans (STEP 1)
4. Stewardship partners
5. Monitoring requirements
6. Land use activities (STEP 1)
7. Stormwater management policies (STEP 8)
8. Construction opportunities and constraints

ANALYSIS

1. Identify funding partners, requirements and funding formulas
2. Identify phasing options and schedule
3. Identify stewardship options
4. Identify monitoring strategies (baseline, during and after construction)
5. Develop an Implementation Plan

STEP 11: Detailed Design

Prepare detailed design drawings and specifications for the SWM facilities and stream restoration works as required.

DATA SOURCES

1. Design relations and criteria from previous STEPS
2. Location of aggregate mines, quarries and disposal sites
3. Transportation route mapping
4. Constraint mapping from previous STEPS
5. Cost estimates from previous STEPS
6. Hydraulic, hydrologic and sediment transport models
7. Monitoring requirements from previous STEPS

ANALYSIS

1. Erosion threshold analysis
2. Plan and cross-section details
3. Bed armor specifications
4. Evaluate scour and deposition scenarios for possible service corridor conflicts
5. Outline a detailed monitoring program for key geomorphic and habitat variables
6. Complete geotechnical analyses of banks as required
7. Relocate services as required
8. Identify construction periods for instream work, access routes, material supply sites, haulage routes, fill disposal areas, etc.

9. Prepare detailed design drawings, landscape plans, specifications and tender documents as required
10. Undertake construction supervision (if required), and
11. Revise cost estimates
12. Implement baseline and during construction monitoring
13. Undertake any other tasks deemed necessary

Note: The above list of tasks and data sources is not exhaustive. Proponents are expected to undertake the design in accordance with their own specifications and requirements as identified by the proponent for any particular project.

APPENDIX C
SIMPLIFIED DESIGN APPROACH

This Appendix provides additional information concerning the derivation and application of the Simplified Design Approach outlined in Section 3.4.3 of the main report. The first sub-section deals with the derivation while the remaining sub-sections elaborate on the three major components of the Simplified Design Approach. These components are:

a) a synoptic level geomorphic survey of the stream channel to collect measurements of channel form and assess channel stability;
b) assessment of the applicability of the Simplified Design Approach for the proposed development; and
c) determination of the volume of source control and storage within an end-of-pipe facility (pond).

This Appendix focuses on the Rapid Geomorphic Assessment and storage volume determination elements.

C.1 Derivation of the Simplified Design Approach

Curves showing pond active storage volume as a function of total amount of directly connected imperviousness area (FRIMP) are provided in Figures C.1(a) and (b) for Soil Conservation Service (SCS) Hydrologic Soils Groups A to B and C to D, respectively. These curves provide a simplified method for the estimation of the active storage volume for small developments (that satisfy the criteria established in Table 3.4), knowing FRIMP, the SCS Hydrologic Soils Group and the amount of Source Control (in this context, source control includes lot level and conveyance controls). The derivation of the approach as outlined below is based on geomorphologic assessments carried out on over 40 streams in Ontario, British Columbia, Texas and Vermont as well as calibration of these curves as presented in Figures C.1(a) and (b) based on a continuous modelling of the flows and erosion potential in two streams in southern Ontario. The two case studies were:

(a) the west branch of the Humber River through the City of Brampton; and
(b) Morningside Tributary through the Town of Markham.

The model used in the analysis was QUALHYMO, a continuous hydrologic simulation model with pond routing algorithms and a routine for the assessment of in-stream erosion potential. The latter is expressed as indices based on a two-dimensional representation of excess boundary shear stress about an arbitrary channel perimeter. The hydrologic component of the model was set up and calibrated to flow gauge data collected by Environment Canada. The erosion index component of the model was set up based on diagnostic geomorphic surveys of the stream channel. The model was calibrated to observed geomorphic activity rates and verified using empirical relations developed for urban streams throughout North America.

Following the setup of the model a corroborative approach was adopted using hydrologic methods (flow exceedance analysis), critical shear stress concepts, and empirical relations and observations of geomorphic activity rates to provide independent but parallel methods of assessment. Different land use conditions were then assessed including:

(i) the pre-development scenario;
(ii) the existing land use condition; and
(iii) the future land use scenario.

The model for the latter two land use conditions was set up to assess the following SWM options:

(a) no SWM measures (baseline condition);
(b) centralized (end-of-pipe) control with no Source Control assuming:
  (i) 2-year control;
  (ii) 25 mm-24 hour control; and
  (iii) Distributed Runoff Control.
(c) centralized control with various levels of Source Control.

In each case the erosion indices were determined and compared to the in-stream erosion criteria adopted for the assessment. The volume of the pond and the pond outlet control structure were adjusted to maximize the reduction of in-stream erosion potential to the maximum amount allowed by the design technique employed. Results from the analysis are presented in MacRae (1996). MacRae (1996) found that the conclusions were consistent among the various methods of assessment. Further, the two case studies are representative of a wide range of stream conditions and hydrographic characteristics found in southern Ontario.

C.2 Synoptic Level Geomorphic Survey

A synoptic geomorphic survey involves:

a) the assessment of channel stability and mode of adjustment; and
b) an engineering-geomorphic survey of the following channel parameters:
  - bankfull channel depth;
  - bankfull channel width;
  - the width of the flood prone area at an elevation corresponding to twice bankfull depth;
  - the composition of the boundary materials composing the:
    i) lower third of the bank (on both banks); and
    ii) the intact bed materials or armor layer.
  - the Soil Conservation Service (SCS) Hydrologic Soil Group within the development.

These parameters will be used in the assessment of the suitability of the Simplified Design Approach for the design of SWM measures for the proposed development, and in the design of the volume of source control and pond storage.

C.3 Rapid Geomorphic Assessment

One approach to the assessment of channel stability and sensitivity to an alteration in the sediment-flow regime is to undertake a Rapid Geomorphic Assessment (RGA) of the channel system. An RGA form, developed for this purpose, is presented as one possible tool (Table C.1).

The RGA form consists of four factors that may be used to suggest evidence of adjustment in channel form or characterize processes indicating mode of adjustment. These factors are:

(a) Evidence of Aggradation (AI);
(b) Evidence of Degradation (DI);
(c) Evidence of Channel Widening (WI); and
(d) Evidence of Planimetric Form Adjustment (PI).

Each of the four factors is represented by a number of indices (see Column (3) in Table C.1). The indices are observed to be present or absent (Columns (4) and (5) in Table C.1). If "present" the index is registered in the "Yes" column and the total number of "Yes" responses is indicated in the cell labelled "Sum of Indices." For example, for the Factor "Evidence of Aggradation," the indices numbered 2, 3, 4 and 5 (Column (2) in Table C.1) were present over a specified length of stream so the "Sum of Indices" would be "4."

The "Factor Value" represents the number of "Yes" responses divided by the total number of responses. Consequently, in the above example, the "Factor Value" would be $AI = 4/7 = 0.57$ (assuming a response of "No" was recorded for all other indices). This process is repeated for each of the Factors listed in Column (1) of Table C.1. The "Factor Values" are then summed and divided by the number of Factors ($m = 4$) to arrive at the Stability Index (SI) value. Experience with approximately 40 streams indicates that the SI value may be interpreted in accordance with criteria outlined in Table C.2.

C.4 Simplified Design Approach: Volume Control

Once it has been established that the Simplified Design Approach is applicable then the volume of source control and the active storage component of the pond may be determined as a function of the SCS Hydrologic Soils Group and total basin imperviousness.

In-Stream Erosion Control Criterion
The change in in-stream erosion potential cannot exceed that change which is equivalent to a 10% paving of the basin without implementation of Stormwater Management measures for the control of erosion potential.

Table C.1: Summary of Rapid Geomorphic Assessment (RGA) Classification

FORM/ PROCESS (1)	GEOMORPHIC INDICATOR		PRESENT		FACTOR
	NO (2)	DESCRIPTION (3)	NO (4)	YES (5)	VALUE (6)
Evidence of Aggradation (AI)	1	Lobate bar			
	2	Coarse material in riffles embedded			
	3	Siltation in pools			
	4	Medial bars			
	5	Accretion on point bars			
	6	Poor longitudinal sorting of bed materials			
	7	Deposition in the overbank zone			
		SUM OF INDICES			
Evidence of Degradation (DI)	1	Exposed bridge footing(s)			
	2	Exposed sanitary/storm sewer/pipeline/etc.			
	3	Elevated stormsewer outfall(s)			
	4	Undermined gabion baskets/concrete aprons/etc.			
	5	Scour pools d/s of culverts/stormsewer outlets			
	6	Cut face on bar forms			
	7	Head cutting due to knick point migration			
	8	Terrace cut through older bar material			
	9	Suspended armor layer visible in bank			
	10	Channel worn into undisturbed overburden/bedrock			
		SUM OF INDICES			
Evidence of Widening (WI)	1	Fallen/leaning trees/fence posts/etc.			
	2	Occurrence of large organic debris			
	3	Exposed tree roots			
	4	Basal scour on inside meander bends			
	5	Basal scour on both sides of channel through riffle			
	6	Gabion baskets/concrete walls/etc. out flanked			
	7	Length of basal scour > 50% through subject reach			
	8	Exposed length of previously buried pipe/cable/etc.			
	9	Fracture lines along top of bank			
	10	Exposed building foundation			
		SUM OF INDICES			
Evidence of Planimetric Form Adjustment (PI)	1	Formation of cute(s)			
	2	Single thread channel to multiple channel			
	3	Evolution of pool-riffle form to low bed relief form			
	4	Cutoff channel(s)			
	5	Formation of island(s)			
	6	Thalweg alignment out of phase meander form			
	7	Bar forms poorly formed/reworked/removed			
		SUM OF INDICES			
STABILITY INDEX (SI) = (AI + DI + WI + PI) / m					

Table C.2: Interpretation of RGA Form Stability Index Value

Stability Index (SI) Value	Classification	Interpretation
SI ≤ 0.2	In Regime	The channel morphology is within a range of variance for streams of similar hydrographic characteristics – evidence of instability is isolated or associated with normal river meander propagation processes
0.21 ≤ SI ≤ 0.4	Transitionally or Stressed	Channel morphology is within the range of variance for streams of similar hydrographic characteristics but the evidence of instability is frequent
SI > 0.4	In Adjustment	Channel morphology is not within the range of variance and evidence of instability is wide spread

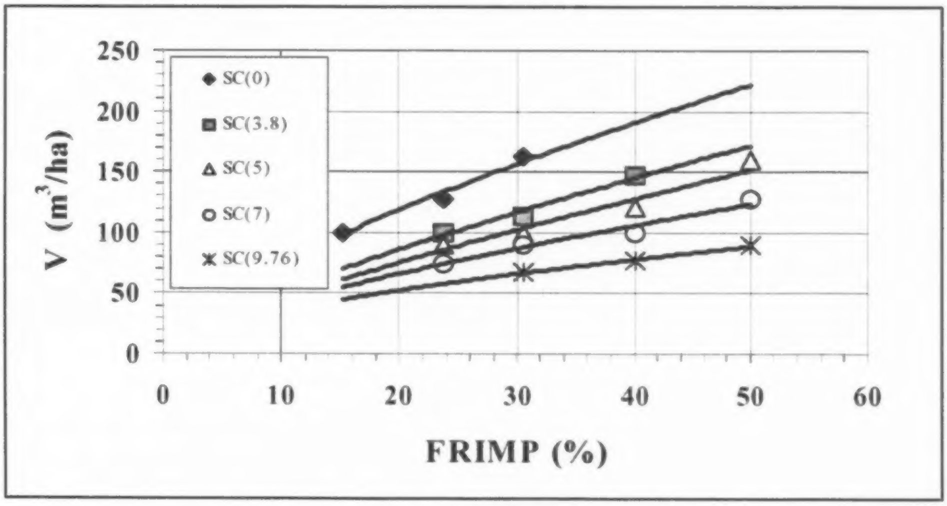
A diminishing return is associated with increasing pond storage for the control of in-stream erosion potential. This appears to be due to:

(a) the loss of effective storage associated with longer flow retention periods as pond size increases and the tendency for precipitation events to occur as multiple events;
(b) the alteration of the hydrologic response of the basin from riverine to lacustrine due to the non-uniform effect of pond attenuation on the distribution of shear stress about the channel boundary (a greater decrease in erosive forces occurs at the bank toe than the channel bed resulting in the tendency to aggrade);
(c) the containment of flows associated with rare flood flow events within the active channel due to peak flow attenuation resulting in the extended duration of high flow rates; and
(d) the impact on the sediment regime increases with larger pond volumes and retention times.

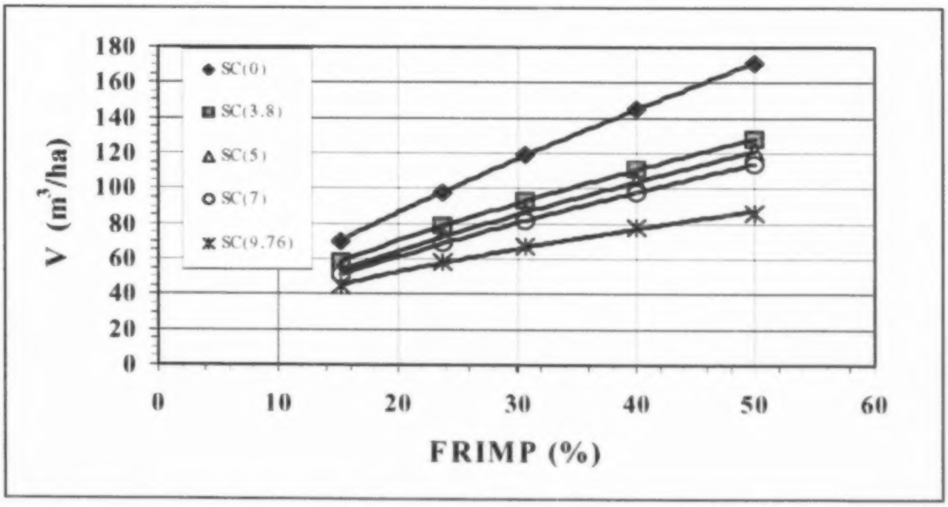
Figure C.1(a) and C.1(b) provide the storage volumes for a given total directly connected basin impervious area (FRIMP) and a range of Source Control (SC) values. As can be seen in Figure C.1(a), an application of 3 watershed-mm of source control will result in a reduction in pond storage volume of approximately 17% for a basin with SCS hydrologic Soil Group 'D' soils and a FRIMP of 40%. It was also observed that the rate of reduction in pond volume with Source Control declines with additional Source Control. Using the previous example, an additional 3 watershed-mm of Source Control would result in an additional decrease in pond storage volume of approximately 10 percent. A further increase in Source Control to a total of 9 watershed-mm would result in an incremental pond storage volume reduction of 5.5 percent, and a total volume reduction of 32%.

Figure C.1: Pond Active Storage Volume for Control of In-Stream Erosion Potential as a Function of Total Directly Connected Impervious Area (FRIMP) and Source Control (including lot level and conveyance control, in watershed-mm)

(a) SCS Soil Groups A and B



(b) SCS Soils Groups C and D



The following steps summarize the approach:

Step 1: Determine the total directly connected impervious area (FRIMP) for the development area.

Step 2: Establish the predominant SCS Hydrologic Soil Group for the development area.

Step 3: Determine the amount of source control practical and feasible for the development area.

Step 4: Based on the FRIMP value, the predominant SCS Hydrologic Soil Group and level of source control select the appropriate curve in Figure C.1 and determine the pond active storage volume for the development area.

Having established the volume requirements for the end-of-pipe and source control measures required to control in-stream erosion potential, the next phase involves determination of the hydraulic performance of the end-of-pipe outlet structure (see Appendix D).

APPENDIX D
DISTRIBUTED RUNOFF CONTROL (DRC) APPROACH

This appendix deals with outlet design for end-of-pipe facilities. The primary objective is to release stored water at a rate which is consistent with meeting established erosion control targets. Several different design approaches may be used; however, this appendix describes only the Distributed Runoff Control (DRC) method.

Under pre-development conditions, the 'effective' flow controlling channel form has been found to be in accordance with bankfull stage (1.5 to 2 year recurrence interval). The smaller mid-bankfull events, although significant in terms of sediment transport within the stream, play a secondary role in the formation of the active channel. Studies have shown that as a result of development, there is an increase in the frequency of occurrence of mid-bankfull flows, and these smaller runoff episodes become the 'effective' geomorphic agents controlling channel form (Figure D.1). Based on these findings, the intent of the DRC approach is the control of in-stream erosion potential for:

a) the range of flows exceeding the critical flow (the rate at which sediment transport of bed forms or intact boundary materials begins), up to bankfull stage, with
b) the highest level of control focussed on flows in the mid-bankfull range.

Flow rates under the critical flow are controlled for water quality purposes while flows exceeding bankfull stage are controlled for flood hazard objectives. The three design zones are illustrated using a conceptualized rating curve for an end-of-pipe facility as shown in Figure D.2. Figure D.2 also illustrates the difference between the rating curves for the:

a) 2 year peak flow shaving method (curve ADF);
b) 25 mm-24 hour approach (curve ABDF);
c) overcontrol procedure (curve AEF); and
d) the Distributed Runoff Control (curve AC2DF) concept.

These curves were developed for a stream formed in boundary materials considered moderately sensitive to scouring (sandy silt to clay loam). Point 'D' in Figure D.2 corresponds to the bankfull flow (Q_{BFL}) defined for the channel at bankfull stage (D_{BFL}). For all flows exceeding Q_{BFL}, flood hazard criteria apply. For all flows less than that corresponding to point 'C1,' water quality criteria apply.

The shaded portion of Figure D.2 denotes the flow rates which correspond to the mid-bankfull stage region of the channel (between 0.5 D_{BFL} and 0.75 D_{BFL}). These are the flows targeted by the DRC method for the greatest level of hydraulic routing. The mean annual flow rate lies within this region, and it is approximated by point 'C2' which is referred to as the DRC 'inflection point.' In more sensitive streams, the inflection point may shift toward point C3. In less sensitive streams, the inflection point may be adjusted toward point C1 as summarized in Table D.1.

Figure D.1: Mid-Bankfull to Bankfull Flow Range and the Corresponding Critical Flows

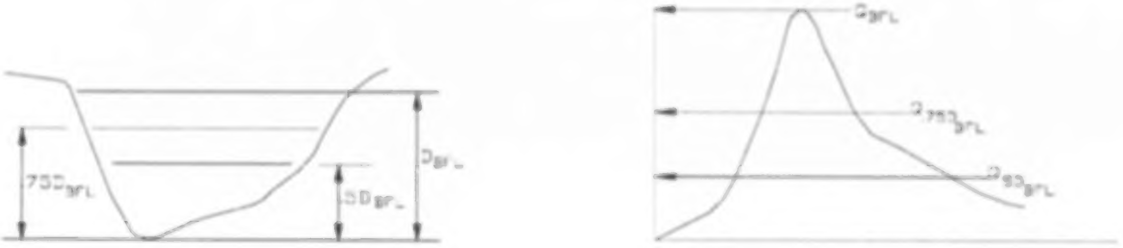

75D_BFL
50D_BFL
D_BFL
Q_BFL
Q_75D_BFL
Q_50D_BFL

Figure D.2: Conceptual Rating Curve for an End-of-Pipe Facility showing:

- (a) 2 Year Peak Flow Shaving Method;
- (b) 25 mm-24 hour Approach;
- (c) Overcontrol Procedure; and
- (d) Distributed Runoff Curve (DRC).

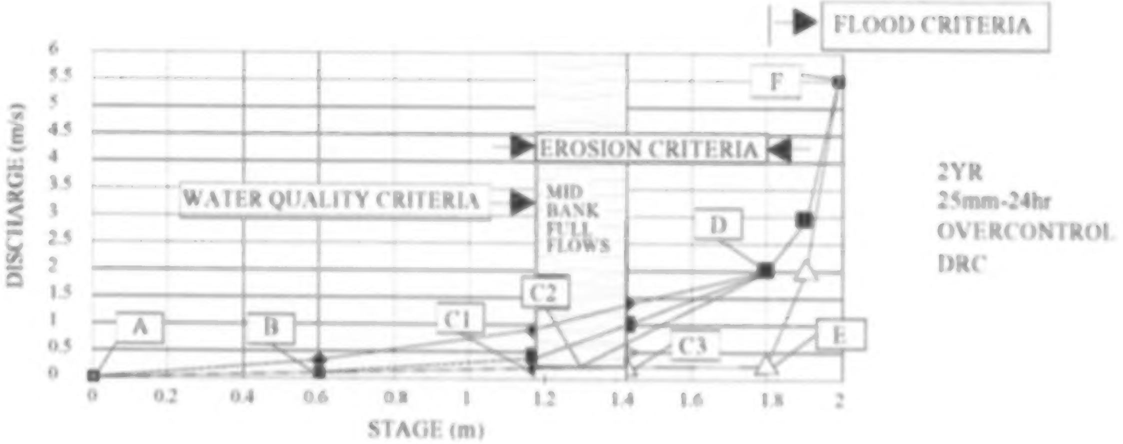


Table D.1: Selection of the DRC Curve Inflection Point

Boundary Material Composition	Inflection Point (Figure D.2)
Sand to Sandy Loam (Very Soft to Soft)	Point C3 defined as Q at 0.75 D_{bfl}
Sandy Silt to Clay Loam (Firm)	Point C2 defined as Q at 0.65 D_{bfl}
Clayey Silt to Silty Clay (Stiff)	Point C1 defined as Q at 0.5 D_{bfl}

The DRC approach follows the overcontrol curve until the DRC inflection point. The overcontrol curve is determined as a multiple of the 2 year peak flow shaving curve. For example, to obtain 80% overcontrol (80%OC), the ordinates for the 2 year peak flow shaving curve are multiplied by 0.2 up to the bankfull flow. The amount of control (e.g., whether it is a 60%OC (multiplier 0.4) or 90%OC (multiplier 0.1)) is determined by the sensitivity of the receiving channel. The more sensitive the channel boundary materials to scour, the greater the degree of control as summarized in Table D.2.

Table D.2: Degree of Overcontrol (Multiplier) as a Function of Boundary Material Composition

Boundary Material Composition	Description	Degree of Overcontrol (Multiplier)
Sand to Sandy Loam	Very Soft (loose to moderately compacted)	0.15-0.2
Sandy Silt to Clay Loam	Soft (moderately compacted)	0.2-0.3
Clayey Silt to Silty Clay	Firm (compacted)	0.3-0.4
Silty Clay	Stiff (highly compacted)	0.4-1.0

The procedure for the development of the DRC rating curve is outlined in the following steps.

Step 1: Determine the composition of the intact boundary material (unless armored in which case the armor layer is used) at the bank toe of both banks (within the range of $0.2D_{bfl}$ to $0.4D_{bfl}$) and within the mid bed region at representative cross-sections in the channel downstream of the point-of-entry of the stormwater drainage from the development site.

Step 2: Using the least resistant of these units, determine the OC multiplier from Table D.1.

Step 3: Construct the 2 year peak flow shaving rating curve (ADF in Figure D.2) by drawing a straight line between points A and D.

Step 4: Construct the OC rating curve by multiplying the ordinates for the 2 year peak flow shaving rating curve by the OC multiplier (ABCE in Figure D.2 in which C is represented by one of C1, C2 or C3).

Step 5: Determine the DRC inflection point from Table D.1.

Step 6: Construct the DRC rating curve (points ABCDF in which C is one of C1, C2 or C3 as determined in Step 5).

APPENDIX E
PLANT SPECIES

Planting Zones

Deep Water > 0.5 m
Shallow Water < 0.5 m
Shoreline Fringe – zone of frequent wetting
Flood Fringe – zone of infrequent wetting
Upland

Scientific Name	**Common Name**
Deep Water	
Brasenia schreberi	Water shield
Ceratophyllum demersum	Coontail
Elodea canadensis	Common waterweed
Lemna minor	Lesser duckweed
Lemna trisulca	Star duckweed
Myriophyllum sibiricum	Northern water milfoil
Myriophyllum verticillatum	Bracted water milfoil
Nuphar variegatum	Yellow pond lily
Nymphaea odorata	White water-lily
Potamogeton gramineus	Variable-leaved pondweed
Potamogeton natans	Floating-leaved pondweed
Potamogeton pectinatus	Sago pondweed
Scirpus validus	Softstem bulrush
Spirodela polyrhiza	Great duckweed
Utricularia vulgaris	Common bladderwort
Vallisneria americana	Tape grass, Eel grass

Note: Choose submergent and floating plants.

Shallow Water

Scientific Name	Common Name
Acorus americanus	Sweet flag
Alisma plantago-aquatica	Water plantain
Calla palustris	Water arum
Carex lacustris	
Carex utriculata	
Equisetum fluviatile	Water horsetail

Scientific Name	**Common Name**
Shallow Water (cont'd)	
Glyceria borealis	Northern manna grass
Polygonum amphibium	Water smartweed
Pontederia cordata	Pickerel weed
Ranunculus reptans	Creeping buttercup
Sagittaria latifolia	Broad-leaved arrowhead
Sagittaria rigida	Stiff arrowhead
Scirpus acutus	Hardstem bulrush
Scirpus fluviatilis	River bulrush
Scirpus pungens	Common three-square
Scirpus validus	Softstem bulrush
Sparganium americanum	American bur-reed
Sparganium eurycarpum	Common bur-reed
Typha angustifolia	Narrow-leaved cattail
Typha latifolia	Broad-leaved cattail
Zizania aquatica	Wild rice

Note: Choose robust, broad-leaved and narrow-leaved plants.

Shoreline Fringe – Near Permanent Pool

Asclepias incarnata	Swamp milkweed
Aster puniceus	Swamp aster
Bidens cernua	Nodding bur-marigold
Calamagrostis canadensis	Canada bluejoint grass
Carex bebbii	
Carex comosa	
Carex crinita	
Carex hystericina	
Carex pseudo-cyperus	
Carex stipata	
Carex stricta	
Carex vulpinoidea	
Cicuta maculata	Water hemlock
Decodon verticillatus	Swamp loosestrife
Dulichium arundinaceum	Three-way sedge
Eleocharis obtusa	Spike rush
Eleocharis smallii	Spike rush
Eupatorium maculatum	Joe pye-weed
Glyceria striata	Fowl manna grass
Iris versicolor	Wild blue flag iris

Scientific Name	**Common Name**
Shoreline Fringe – Near Permanent Pool (cont'd)	
Juncus articulatus	Jointed rush
Juncus balticus	Baltic rush
Juncus canadensis	Canada rush
Juncus effusus	Soft rush
Juncus pelocarpus	Brown-fruited rush
Juncus torreyi	Torrey's rush
Leersia oryzoides	Rice cut-grass
Lobelia cardinalis	Cardinal flower
Lycopus americanus	Water horehound
Lysimachia terrestris	Swamp candles
Mimulus ringens	Monkey flower
Osmunda regalis	Royal fern
Phalaris arundinacea	Reed canary grass
Potentilla palustris	Marsh cinquefoil
Rumex orbiculatus	Great water dock
Scirpus atrovirens	Green bulrush
Scirpus cyperinus	Wool grass bulrush
Scirpus pendulus	Pendulus bulrush
Scutellaria galericulata	Marsh skullcap
Sium sauve	Water parsnip
Thelypteris palustris	Marsh fern
Triadenum fraseri	Marsh St. John's Wort
Shrubs	
Alnus incana	Speckled alder
Cephanlanthus occidentalis	Buttonbush
Cornus stolonifera	Red osier dogwood
Ilex verticillata	Winterberry
Lonicera oblongifolia	Swamp fly honeysuckle
Myrica gale	Sweet gale
Nemopanthus mucronatus	Mountain holly
Rhamnus alnifolia	Alder-leaved buckthorn
Ribes triste	Swamp red currant
Rosa palustris	Swamp rose
Rubus pubescens	Dwarf raspberry
Salix bebbiana	Beaked Willow
Salix exigua	Sandbar willow
Salix lucida	Shining willow
Salix petiolaris	Slender willow
Salix pyrifolia	Balsam willow
Spirea alba	Meadowsweet

Scientific Name	**Common Name**
Shoreline Fringe – Near Permanent Pool (cont'd)	
Trees	
Acer saccharinum	Silver maple
Fraxinus nigra	Black ash
Quercus bicolor	Swamp white oak
Salix nigra	Black willow
Shoreline Fringe – Near Flood Fringe	
Aster novae-angliae	New England aster
Aster umbellatus	Flat topped aster
Bidens frondosa	Common beggar-ticks
Cyperus esculentus	Yellow nutsedge
Equisetum arvense	Field horsetail
Eupatorium perfoliatum	Boneset
Impatiens capensis	Spotted touch-me-not
Impatiens pallida	Pale touch-me-not
Juncus tenuis	Path rush
Lilium michiganense	Michigan lily
Lysimachia ciliata	Fringed loosestrife
Osmunda cinnamomea	Cinnamon fern
Urtica dioica	Stinging Nettle
Vines	
Echinocystis lobata	Wild cucumber
Vitis riparia	Riverbank grape
Shrubs	
Aronia melanocarpa	Black chokeberry
Cornus foemina	Grey dogwood
Lindera benzion	Spicebush
Physocarpus opulifolius	Ninebark
Potentilla fruticosa	Shrubby cinquefoil
Ribes americanum	Wild black currant
Rubus idaeus	Wild red raspberry
Salix amygdaloides	Peach-leaved willow
Salix discolor	Pussy willow
Salix eriocephala	Woolly headed willow
Sambucus canadensis	Elderberry
Vaccinium myrtilloides	Velvet-leaf blueberry
Viburnum cassinoides	Northern wild raisin
Viburnum trilobum	Highbush cranberry

Scientific Name	**Common Name**
Shoreline Fringe – Near Flood Fringe (cont'd)	
Trees	
Abies balsamea	Balsam fir
Carya laciniosa	Shellbark hickory
Fraxinus pennsylvanica	Red ash
Larix laricina	Tamarack
Picea mariana	Black spruce
Platanaus occidentalis	Sycamore
Populus balsamifera	Balsam poplar
Quercus palustris	Pin oak
Thuja occidentalis	Eastern white cedar
Ulmus americanum	American elm
Flood Fringe	
Vines	
Clematis virginiana	Virgin's bower
Menispermum canadense	Canada moonseed
Parthenocissus quinquefolia	Virginia creeper
Smilax hispida	Bristly greenbrier
Shrubs	
Crataegus crus-galli	Cockspur thorn
Lonicera hirsuta	Hairy honeysuckle
Prunus virginiana	Choke cherry
Viburnum lentago	Nannyberry
Trees	
Acer rubrum	Red maple
Betula alleghaniensis	Yellow birch
Carya cordiformis	Bitternut hickory
Populus deltoides	Eastern cottonwood
Quercus macrocarpus	Bur oak

Many of the species listed under Shoreline Fringe – Near Flood Fringe may be appropriate near the inside edge of the flood fringe. Flooding near the outside edge of the zone may be extremely rare such that the conditions for upland species will exist. The listed species are tolerant of intermediate moisture conditions.

Scientific Name	**Common Name**
Upland	
Trees	
Acer saccharum	Sugar maple
Betula papyrifera	Paper birch
Crataegus spp.	Hawthorn
Fraxinus americana	White ash
Juniperus virginiana	Eastern red cedar
Pinus banksiana	Jack pine
Pinus strobus	Eastern white pine
Populus tremuloides	Trembling aspen
Quercus alba	White oak
Quercus rubra	Red oak
Tsuga canadensis	Eastern hemlock
Shrubs	
Acer pensylvanicum	Striped maple
Amelanchier alnifolia	Service-berry
Amelanchier arborea	Juneberry
Amelanchier sanguinea	Round-leaved serviceberry
Amelanchier spicata	Shadbush serviceberry
Arctostaphylos uva-ursi	Bearberry
Ceanothus americanus	New Jersey tea
Cornus rugosa	Round-leaved dogwood
Corylus americana	American hazelnut
Corylus cornuta	Beaked hazelnut
Diervilla lonicera	Bush honeysuckle
Hamamelis virginiana	Witch hazel
Lonicera dioica	Wild honeysuckle
Prunus pensylvanica	Pin cherry
Ribes cynosbati	Prickly gooseberry
Rhus aromatica	Fragrant sumac
Rhus typhina	Staghorn sumac
Rosa blanda	Smooth wild rose
Rubus allegheniensis	Common blackberry
Salix humilis	Upland willow
Sambucus racemosa	Red-berried elder
Shepherdia canadensis	Buffalo-berry
Symphoricarpos albus	Snowberry
Viburnum acerifolium	Maple-leaved viburnum
Viburnum rafinesquianum	Downy arrow-wood
Zanthoxylum americanum	Prickly ash

APPENDIX F
INFILL DEVELOPMENT AND WATERSHED REHABILITATION PLAN

The development of an *Infill Development* or *Subwatershed Rehabilitation Plan* (terms used interchangeably in this appendix) is the preferred approach in addressing stormwater quality and quantity concerns associated with infill development.

A plan is particularly important for larger infill sites (> 5 ha); in municipalities where significant growth is expected from infill development; and for effective use of off-site systems (OSS) stormwater management practices because:

- a wider range of SWMPs may be applied within the infill site for larger sites and off-site systems;
- the potential impact on the receiving environment will likely be more significant; and
- the opportunity for restoring existing environmental problems within the tributary area are more feasible.

Chapter 5 outlines some options that may be considered for small infill development (< 5 ha) where anticipated infill development is not sufficient to warrant the preparation of an Infill Development/Subwatershed Rehabilitation Plan.

The intent of this appendix is to provide direction/general steps in developing an Infill Development/Subwatershed Rehabilitation Plan. Many of these steps are similar to those outlined for environmental planning studies (Chapter 2) and retrofit studies (Appendix G). The major difference from environmental planning studies is that infill developments occur in built-up areas and impacts on the receiving water may already be occurring.

Figure F.1 illustrates a hypothetical site which will be used to assist in defining the steps that need to be undertaken. This large infill site is assumed to be located within a developed area serviced by storm sewers which discharge to a small stream which is a tributary of a larger stream.

Major Steps in Developing an Infill Development/Subwatershed Plan

There are three major steps in developing an Infill Development/Subwatershed Plan:

Step 1: Develop Environmental Goals, Objectives and Targets

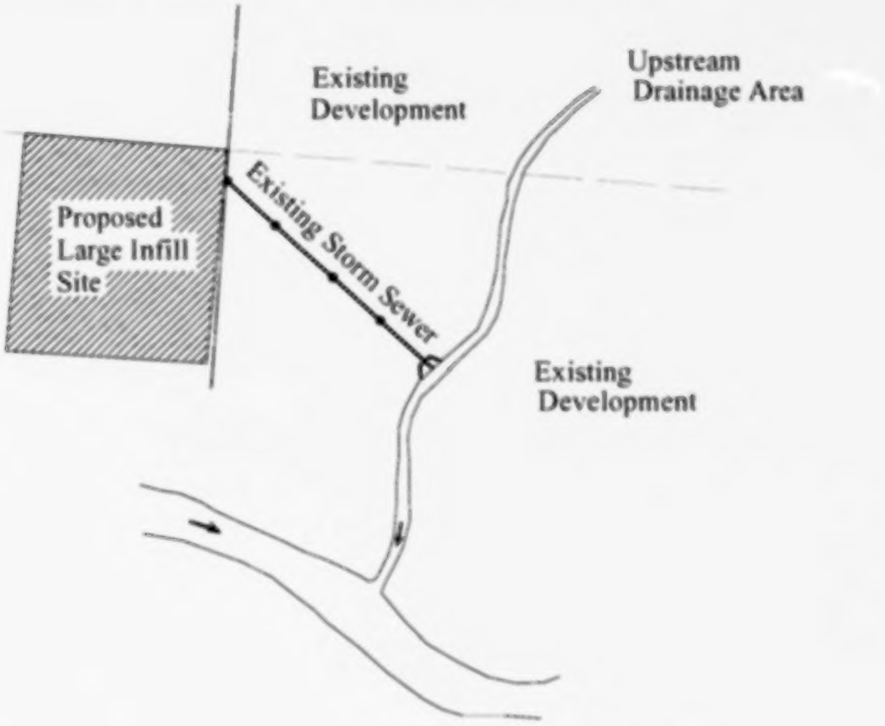
Step 2: Undertake Technical Studies

Step 3: Identify and Select Preferred SWMP

Step 1: Develop Environmental Goals, Objectives and Targets

The environmental goals, objectives and targets may either be available from previous studies or would need to be developed as part of the Plan once the technical studies have been completed. Chapter 2 and Appendix G provide direction for developing environmental goals, objectives and targets.

Figure F.1: Hypothetical Example for Proposed Large Infill Site


Existing Development
Upstream Drainage Area
Proposed Large Infill Site
Existing Storm Sewer
Existing Development

Step 2: Undertake Technical Assessments

A variety of field/technical studies may be required in order to define existing environmental conditions; assess opportunities and constraints; and assist in identifying SWMPs which are suitable based on site conditions as well as the defined environmental goals, objectives and targets.

The general types of component studies may include:

- aquatic;
- surface water quantity and quality;
- groundwater;
- geomorphologic;
- terrestrial; and
- infrastructure.

A brief overview of key points to be considered for component studies are provided below:

Aquatic
Aquatic communities (particularly fish species) are typically used as an indicator of environmental health. Section 1.3 of this manual discusses the impact of stormwater runoff on stream ecosystems. The four factors identified (i.e., changes in hydrology, changes in urban stream morphology, changes in stream water quality and changes in stream habitat and ecology) all generally impact existing aquatic communities.

Table F.1 lists one approach for defining the hydrologic, morphologic, water quality and habitat requirements for a range of different aquatic communities. This table may be used to assist in defining aquatic objectives for the target species and defining required standards to meet an aquatic objective. The integrated set of standards that are required may, in turn, be compared to actual physical and biological conditions in order to identify the performance standard(s) which are limiting.

Other considerations such as the identification of physical barriers together with benthic invertebrate work may well be required to completely address aquatic goals and objectives.

Surface Water Quantity and Quality
For surface water quantity there are three general conditions that need to be considered, including:

- low flows (baseflow);
- frequent flows (generally associated with erosion); and
- high (flooding) or infrequent flows.

Baseflow within the stream generally needs to be determined since lack of baseflow impacts aquatic communities and may also indirectly impact water quality conditions. Frequent flows and high flows are generally derived via a modelling exercise (see geomorphologic sub-section). For high flows, a hydrologic/hydraulic assessment may be required in order to determine the impact on downstream areas and, therefore, the requirement for flow controls for the proposed site.

The approach for undertaking water quality assessments is changing. Whereas past efforts focussed on collecting wet weather samples at a number of sites, present efforts are considering:

- replacing wet weather sampling with comprehensive water quality sampling programs with streamlined quality sampling programs together with programs focussing on biologic indicators (biomap, benthic invertebrate); and

- monitoring dry weather conditions as well as wet weather conditions as urban streams typically have short periods (1 - 3 hours out of 72 hours on average) when wet weather flows govern and contaminant levels during dry weather have been found to be higher than initially thought.

Table F.1: Biophysical Performance Standards for Aquatic Ecosystem Objectives

	AQUATIC ECOSYSTEM OBJECTIVE			
Aquatic Performance Standard	**□ Brook Trout**	**□ Brook Trout** **□ Brown Trout**	**□ Pike** **□ Darters** **□ Sunfish**	**□ Longnose Dace** **□ Brown Bullhead** **□ Brook Stickleback**
HYDROLOGY • baseflow	• minimum 30% of average annual daily flow	• minimum 10-20% of average annual daily flow	• minimum 5% of average annual daily flow or sufficient to maintain isolated pools	• minimum to maintain isolated pools, may be < 5% of average annual daily flow
• bankfull frequency	• 1-2 times per year	• 1-2 times per year	• as required to protect downstream aquatic communities	• as required to protect downstream aquatic communities
CHANNEL MORPHOLOGY	• dynamically stable channels with 'natural' features	• dynamically stable channels with 'natural' features	• dynamically stable channels with 'natural' features	• dynamically stable channels with 'natural' features
• average pool area as % of total surface area at low flow	• > 12%	• > 4%	• > 4%	• > 4%
• average riffle area as % of total surface area at low flow	• > 12%	• > 10%	• generally > 5%	• may be < 5%
• average minimum summer pool depth	• 0.5 m	• 0.3 m	• 0.2 m	• 0.2 m
• bankfull width-to-depth ratio	• generally < 10	• generally 5-10	• generally 5-10	• no requirement

Table F.1: Biophysical Performance Standards for Aquatic Ecosystem Objectives (cont'd)

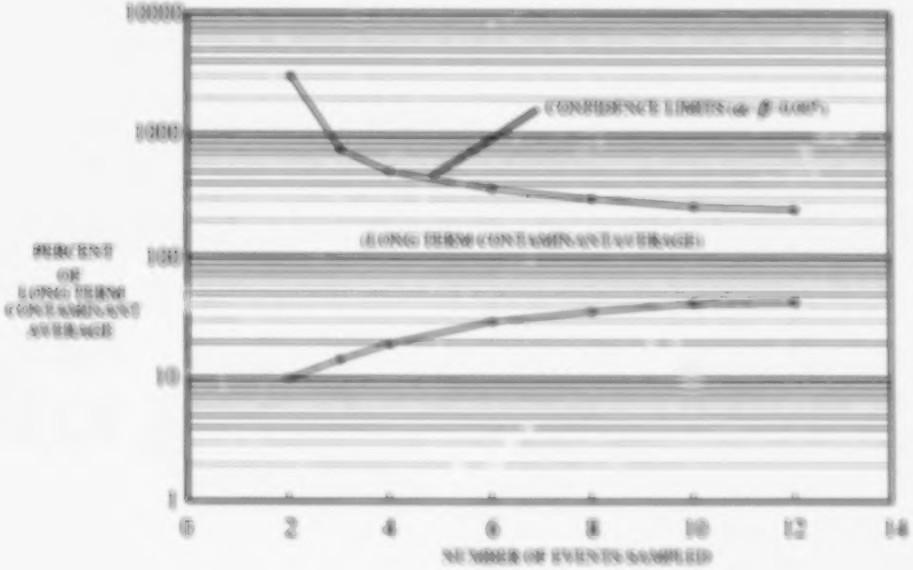
Aquatic Performance Standard	AQUATIC ECOSYSTEM OBJECTIVE			
	□ Brook Trout	□ Brook Trout □ Brown Trout	□ Pike □ Darters □ Sunfish	□ Longnose Dace □ Brown Bullhead □ Brook Stickleback
IN-STREAM COVER	• minimum total in-stream cover 30-40% by surface area • woody debris presents up to 10% of surface area • minimum 15% of surface area with overhead cover • minimum 5% of surface area with overhead cover	• minimum 10% of stream area during low flow • 10-20% of bottom of pool/backwater habitats covered by logs, vegetation, woody debris and boulder • cover at stream margins critical for juvenile fish	• minimum 5% of stream area during low flow • 10% of bottom of pools/ breakwaters covered by logs, vegetation, woody debris and boulder	• minimum 5% of stream area during low flow • 5% of bottom of pools/ backwaters covered by logs, vegetation, woody debris and boulder
SUBSTRATE	• well-sorted riffle zones • maximum 25% fines in spawning substrates • maximum 30% fines in riffle zones • upwelling conditions required • minimum 50% of riffles composed of cobble, rubble, small boulder	• D5O in pools generally < 80 mm • fines in riffle zones moderate to low (< 50%)	• D5O in pools generally < 40 mm • more fines in riffle zones, generally > 50%	• no requirement

Table F.1: Biophysical Performance Standards for Aquatic Ecosystem Objectives (cont'd)

Aquatic Performance Standard	AQUATIC ECOSYSTEM OBJECTIVE			
	▫ Brook Trout	▫ Brook Trout ▫ Brown Trout	▫ Pike ▫ Darters ▫ Sunfish	▫ Longnose Dace ▫ Brown Bullhead ▫ Brook Stickleback
RIPARIAN HABITAT				
• shaded during 1000-1400hr	• minimum 35%	• minimum 0% • maximum 50-75%	• minimum 0% • maximum 75%	• minimum 0% • maximum 100%
• woody debris	• important component of in-stream cover and roughness	• important for roughness and refuge during peak flows	• woody debris less important	• less important
WATER QUALITY				
• maximum annual water temperature	22°C	30°C	31-35°C	31-35°C
• average annual total suspended solids (ppm)	< 20	< 150	< 200	< 400
• dissolved oxygen (ppm)	> 5	> 3-4	> 2	< 2
• spills	none	none	none	none
BARRIERS	• remove as feasible	• removal as feasible	• minimize as feasible	• minimize as feasible

Furthermore, in cases where wet weather sampling is being undertaken, at least eight events are being sampled in order to reasonably define the chemical constituents over a variety of rainfall conditions (see Figure F.2).

Figure F.2: Comparison of Sample Contaminant Averages to Long-Term Contaminant Average



Groundwater
Key tasks to be undertaken include defining basic geologic conditions, identifying recharge/discharge areas and determining the relative importance of the site with respect to protecting groundwater supply; and determining a water budget for the proposed infill site under present and proposed conditions. With respect to the last point, the water budget assessment presented in Section 3.2 may be useful.

Geomorphologic
Section 3.4 together with Appendices B through D provide information with respect to erosion and geomorphologic assessments. Several key points that must be considered when undertaking these assessments are outlined below.

Typically, a stream will take a considerable time (25 to 60 years) to respond to land use change. Therefore, depending upon the relative timing between the proposed infill site and previous development, the stream may still be enlarging or have already enlarged to its ultimate cross-sectional shape.

Stream channels enlarge at a different rate depending upon the total basin imperviousness value. Therefore, the stream will respond to a different degree depending upon the relative size of the proposed infill site to the total catchment area and the relative level of development (or percent imperviousness) within the basin.

A majority of urban streams have been altered over time. Alteration may have taken the form of the physical relocation of a stream, construction of a roadway across the stream or modification of the connectivity between the low flow channel and the floodplain. As a result of the alterations as well as the ongoing cross-sectional changes that are occurring, urban streams typically are subject to excessive erosion rates and have lost many of the attributes that are necessary to provide habitat for sensitive aquatic species. If improving aquatic conditions, protecting public property or restoring recreational/environmental opportunities along the stream corridor are goals as set out in the study, then restoration of the stream will likely be needed.

Terrestrial

Terrestrial resource assessments typically include wetlands, woodlots, landforms and specially designated natural areas. An approach for undertaking terrestrial assessments within an existing developed area is not covered in this appendix. Assessment of the proposed infill site will be necessary to ensure that the above noted resources are protected.

Infrastructure

An assessment of the existing storm sewer system from the proposed infill site to the receiving stream may be required depending upon capacity constraints, the proposed release rate of flows from the infill site and the potential for basement flooding in areas within the sewershed. Accommodation of major system flow (Section 4.7.2) must also be accounted for.

Step 3: Identify and Select Preferred SWMP(s)

Once the environmental goals, objectives and targets have been confirmed and the technical studies completed, the preferred SWMP can be identified and selected. Generally, a combination of practices will be required to address the overall environmental targets. Table 1.3 summarizes different SWMPs and their suitability with respect to different environmental criteria (e.g., water quality, erosion, water quantity). This table should be used in conjunction with Table 4.1 which summarizes physical criteria that need to be considered when evaluating each type of SWMP.

As discussed in Chapter 5, on-site stormwater management is generally the preferred option in addressing cumulative stormwater impacts; however, in certain situations it may be ineffective or impractical because of physical constraints. In these cases, an off-site system (OSS) SWMP may be considered at another location within the same subwatershed and could be financed through a financial contribution from the project proponent based on formulas developed by local municipalities. OSS are most effective within the context of an Infill Development/ Subwatershed Rehabilitation Plan.

Besides off-site SWMPs, municipalities may also be able to use funds for watershed management and restoration works. For example, the technical assessment may find that some stream reaches lack suitable habitat for a target aquatic community and are experiencing ongoing erosion problems as the channel continues to enlarge. Furthermore, construction of erosion control measures on site will result in no net increase in erosion potential but will not restore the degraded habitat conditions or prevent ongoing erosion from existing development.

In this case, construction of an in-stream works to improve habitat conditions and curtail ongoing erosion processes could be considered rather than the construction of on-site stormwater erosion control measures. However, before this approach can be used, there must be concurrence from the appropriate agencies and the private sector. Furthermore, all existing policies, guidelines and acts must be reviewed.

APPENDIX G
METHODOLOGY FOR EVALUATING RETROFIT OPTIONS/RETROFIT STUDIES

G.1 Introduction

Retrofitting of existing infrastructure may be required to achieve water balance, water quantity, water quality, and erosion and flood control goals. The objective of this appendix is to outline a methodology that can be used to prepare a stormwater retrofit study which evaluates retrofit options.

The term "retrofit" is used in a general sense and includes retrofitting of:

- existing SWM practices in order to provide multiple benefits (e.g., retrofitting an existing dry pond which presently provides only a flood control function to a multi-purpose facility providing baseflow augmentation, water quality, and erosion and flood control functions);

- infrastructure along a roadway (in order to better reproduce the historical water budget or reduce water quality loadings); and

- an area (from as small as a municipal block to as large as a subwatershed) in order to achieve environmental goals and targets (e.g., reduction of in-stream phosphorus levels to meet Provincial Water Quality Objectives (PWQO)).

G.2 Background

Initially, retrofitting was geared towards water quantity and water quality issues. For example, many municipalities completed Pollution Control Plans which involved retrofitting of infrastructure to address concerns such as: basement flooding, combined sewer overflows, and parameters which exceeded PWQOs. Retrofitting to address environmental concerns, such as loss of aquatic habitat, excessive rates of erosion, diminishing baseflows or loss of natural features, is a more recent occurrence.

Early retrofit studies tended to examine entire watersheds (e.g., Don River); summarize the environmental concerns; and identify a range of SWM practices which if implemented could improve existing environmental conditions. More recently, retrofitting opportunities are being identified within subwatershed studies or environmental studies undertaken by municipalities or regions.

G.3 Methodology for Evaluating Retrofit Options

The following methodology/steps could be used in selecting the preferred retrofit option(s):

Step 1: Define Environmental Goals, Objectives and Targets

Step 2: Identify General Types of Suitable SWM Practices based on Environmental Goals, Objectives and Targets

Step 3: Undertake Technical Assessment

Step 4: Select SWM Practices Based on Evaluation Criteria

Step 5: Develop an Implementation Plan

Step 1: Define Environmental Goals, Objectives and Targets

In order to define the environmental goals, objectives and targets, an understanding of current and potential future environmental conditions is needed. This information may be available from existing studies, or may require interpretation of available information together with a field program.

Following this task, an assessment of the inter-relationships between the environmental resources needs to be made as does the factors characterizing the health of the resources (Table G.1), and the identification of key ecologic constraints and opportunities (see Chapter 2 for further details). Environmental goals, objectives and targets may then be defined.

The environmental goals, objectives and targets provide the framework for Steps 2 to 5. The goals, objectives and targets may vary from relatively straightforward to complex. For instance, a goal of reducing in-stream phosphorus concentrations by an average of 20 percent is fairly straightforward, whereas a goal of improving a degraded ecosystem with one that supports a healthy warm water fishery, provides stable flow regimes and results in minimal exceedences of PWQOs for key water quality constituents is fairly complex.

Step 2: Identify General Types of Suitable SWM Practices (Qualitative Screening Based on Environmental Goals, Objectives and Targets)

An initial qualitative screening of potential SWM practices early in the process (prior to other assessments, e.g., technical feasibility or costs) is useful to identify SWM practices that would likely meet the environmental goals established in Step 1 as well as identifying potential conflicts.

Table G.1: Factors Characterizing the Quality and Quantity of Environmental Resources

	FLOW FACTOR			STREAM QUALITY/ GEOMORPHIC FACTORS					ENVIRONMENTAL QUALITY PARAMETERS								HABITAT BUFFER FACTORS		
ENVIRONMENTAL RESOURCES	Water Balance	Peak Flow	Baseflow	Width/Depth	Stream Gradient	Riparian Cover	In-Stream Cover	Bedload	TSS	Nutrients	Heavy Metals	Organics	Fecal Bacteria	Toxicity Test	Colour	Temperature	Habitat Diversity and Sensitivity	Width of Buffer	Presence of Endangered Species
I. GREAT LAKES ECOSYSTEM																			
Great Lakes Water Quality	×	×	×			×	×	×	×	×	×	×	×	×	×	×			
II. ROUGE RIVER SURFACE WATER SYSTEM																			
Flowing Water	×	×	×																
Surface Water Quality	×	×	×			×	×	×	×	×	×	×	×	×	×	×			
Aquatic Sediments								×		×	×	×		×	×				
Benthic Organisms		×	×		×	×	×	×	×	×							×		
III. PUBLIC HEALTH																			
Drinking Water – Groundwater	×								×	×	×	×	×	×	×	×			
Edible Fish											×	×		×	×				
Contact Recreation	×		×						×				×						
IV. GROUNDWATER																			
Recharge/Discharge Areas	×		×														×	×	
Groundwater Quality			×							×	×	×	×	×	×	×			

Table G.1: Factors Characterizing the Quality and Quantity of Environmental Resources (cont'd)

	FLOW FACTOR			STREAM QUALITY/ GEOMORPHIC FACTORS						ENVIRONMENTAL QUALITY PARAMETERS						HABITAT BUFFER FACTORS			
ENVIRONMENTAL RESOURCES	Water Balance	Peak Flow	Baseflow	Width/Depth	Stream Gradient	Riparian Cover	In-Stream Cover	Bedload	TSS	Nutrients	Heavy Metals	Organics	Fecal Bacteria	Toxicity Test	Colour	Temperature	Habitat Diversity and Sensitivity	Width of Buffer	Presence of Endangered Species
V. PUBLIC SAFETY																			
Flooding	×	×			×														
Erosion	×	×			×	×	×	×	×										
VI. AQUATIC COMMUNITIES																			
Community Diversity	×	×	×	×	×	×	×	×	×	×				×		×	×		
Habitat	×	×	×	×	×	×	×	×	×	×						×	×	×	×
VII. TERRESTRIAL FEATURES																			
Wetlands	×	×	×			×			×	×	×	×					×	×	×
Woodlots	×																×	×	×
Valleylands	×				×	×	×										×	×	×
VIII. WILDLIFE																			
Wildlife Communities	×				×	×											×		
Wildlife Habitats	×				×	×											×	×	×
IX. AESTHETICS	×	×			×	×	×		×										
X. RECREATION		×	×			×	×												

Source: Best Management Practices Environmental Resource Management Project – Town of Markham, 1996.

Table G.2 could be used in an initial screening to qualitatively assess whether or not the SWM Practices/Watershed Management Practices outlined (horizontal axis) would improve a given environmental resource (environmental goals), potentially result in conflict, or would likely have a strong potential for conflict with environmental goals.

This initial screening provides indication of the potential for the various SWM Practices/ Watershed Management Practices to result in the most benefit (as indicated by a large number of potential for improvement "X") or result in conflicts (as indicated by a large number of potential conflict "O"; and strong potential for conflict "~") (see Table G.2).

This assists decision-makers in selecting a list of potential SWM practices. It does not, however, directly lead to the inclusion or exclusion of a given SWMP. This type of table format may also be a useful tool in presenting options to the public.

Step 3: Undertake Technical Assessment

Steps 1 and 2 provide key environmental goals/objectives/targets and an initial qualitative indication as to which SWM practices are likely to be the most effective in meeting these goals. Step 3 involves undertaking a technical assessment in order to determine which SWM practices or group of SWM practices, when implemented, would assist in meeting these goals. More than one set of alternatives needs to be identified since further assessment with respect to technical feasibility, cost, etc., is required.

The technical assessment method used will depend on the situation. For example, to retrofit a series of dry ponds in order to meet specific in-stream water quality conditions, a relatively straightforward assessment utilizing water quantity and water quality models may be used. Alternatively, if enhancement of aquatic habitat conditions together with improvements in stream stability are the objectives, then a variety of tools, including habitat, geomorphologic and water resource models, may be required.

Step 4: Select SWM Practices Based on Evaluation Criteria

Step 3 generally identifies several technically feasible SWM options. For example, any combination of ponds in a series of existing dry ponds could be retrofitted in order to meet the required water quality objectives. Various combinations of source control measures, pond retrofits or stream rehabilitation could be undertaken in order to enhance aquatic habitat conditions and stabilize a stream.

Step 4 involves evaluating each of the feasible options against series of criteria and ultimately selecting the preferred option. Examples of evaluation criteria are provided in a number of documents including the Municipal Environmental Assessment which uses natural, social and

Table G.2: Environmental Resources Improved by or Potentially Impacted by SWM Practices/Watershed Management Practices

	SWM Practices/Watershed Management Practices						
ENVIRONMENTAL RESOURCES	Storage Water Quality/ Q Pond	Infiltration Devices	Artificial Wetlands	Urban Retrofit	Riparian Buffer Creation (Valleyland Reforestation)	Aquatic Habitat Restoration	Groundwater Recharge Protection
I. GREAT LAKES ECOSYSTEM							
Great Lakes Water Quality	×	×		×			
II. ROUGE RIVER SURFACE WATER SYSTEM							
Flowing Water		×	×				×
Surface Water Quality	×	×		×			
Aquatic Sediments	×					×	
Benthic Organisms	×		×		×	×	
III. PUBLIC HEALTH							
Drinking Water – Groundwater	○	~					×
Edible Fish	×	×		×			
Contact Recreation							
IV. GROUNDWATER							
Recharge/Discharge Areas		×	×		×		×
Groundwater Quality		~	○				×
V. PUBLIC SAFETY							
Flooding	×	×	×				
Erosion	×	×	×				
VI. AQUATIC COMMUNITIES							
Community Diversity				○	×	×	×
Habitat	×, ~	×			×	×	×
VII. TERRESTRIAL FEATURES							
Wetlands	×		×		×	×	
Woodlots					×		
Valleylands					×	×	
VIII. WILDLIFE							
Wildlife Communities					×		
Wildlife Habitats					×		
IX. AESTHETICS	×	×	×	×	×	×	
X. RECREATION	×	×			×		

× Potential for Improvement ○ Potential Conflict ~ Strong Potential for Conflict

Source: Best Management Practices Environmental Resource Management Project – Town of Markham, 1996.

economic criteria as the basis for selecting the preferred alternative. Evaluation criteria should generally consider:

- public acceptance;
- cost – capital as well as operation and maintenance;
- land requirements with respect to associated impact on present/future land uses;
- implementability of option; and
- potential for environmental improvement.

Step 5: Develop an Implementation Plan

Once the preferred alternative has been selected, an Implementation Plan needs to be developed. For straightforward initiatives, implementation may only require addressing funding issues and identifying the agency responsible for overseeing construction.

For more involved projects, a series of decisions may need to be made, including:

- deciding whether or not an implementation committee needs to be established and defining the committee's role;

- defining lead and secondary agencies responsible for implementation, funding alternatives, and policy considerations for each of the proposed SWM practices;

- prioritizing proposed SWM practices – generally based on cost-effectiveness, ease of implementation, and on the provision that considerable improvement in environmental conditions be implemented first;

- defining education programs, the role of the public and stewardship opportunities; and

- defining long-term monitoring requirements to define the effectiveness of the measures to meet the environmental goals, objectives and targets.

G.4 Methodology for Evaluating Retrofit Options – Town of Markham Case Study

The previous sections provided general information on a five-step methodology for evaluating retrofit options. The "Town of Markham Stormwater Retrofit Study" was completed by the Toronto and Region Conservation Authority and the Town of Markham in 1999. Provided below is a summary of the Town of Markham findings using the five-step methodology to evaluate its retrofit options.

Background
The objective of the Town of Markham retrofit study was to prioritize the retrofit of eleven stormwater management ponds in terms of water quality and erosion control. The study area is located within the Town of Markham and the ponds are located within urbanizing areas between Highway 404, Highway 48, Steeles Avenue and Major MacKenzie Drive. The ponds are scattered along the upper reaches of the Rouge River and a number of tributaries, including German Mills Creek, Beaver Creek, Burndenette Creek and Robinson Creek. Table G.3 summarizes the pond number, name and type, as well as land use within the catchment area.

During the course of the study, a comprehensive screening and prioritization protocol was developed in order to assess the retrofit potential of the ponds. The protocol incorporated logistical constraints (e.g., adjacent land uses and space for enlargement), as well as the following three environmental components:

- ecological significance of the receiving stream;
- potential erosion control benefit; and
- potential water quality benefit.

The water quality and geomorphologic approaches outlined in Chapter 3 and Appendices B through D were also used to assess options.

Step 1: Define Environmental Goals, Objectives and Targets

The major goal/objective of the Markham study was to determine the potential for maintaining/restoring the environmental conditions of stream tributaries by retrofitting existing ponds to address water quality and erosion concerns.

Step 2: Identify General Types of Suitable SWM Practices (Qualitative Screening Based on Environmental Goals, Objectives and Targets)

A qualitative screening of different types of SWM practices was not undertaken because the study objective was to assess only one type of SWM practice, i.e., existing stormwater management ponds in the Town of Markham.

Step 3: Undertake Technical Assessment

The technical assessment was geared for meeting the environmental goal/objective described in Step 1 and for providing information that could be used for Step 4. The study was undertaken at a planning level; therefore, certain technical findings needed to be assessed in greater detail. As part of the technical assessment, the following were determined:

A. habitat index for the streams;
B. erosion control benefit of each pond; and
C. water quality benefit of each pond.

A. Habitat Index (HI)

A Habitat Index was determined for each stream based on previous field work studies. HI values range from one (low sensitivity) to five (high sensitivity). Stormwater ponds flowing to a stream highly sensitive to environmental impacts were considered to be a higher retrofit priority than ponds flowing to a stream with a lower sensitivity.

B. Erosion Control Benefit of Each Pond

The erosion control benefit of each pond was estimated by initially comparing the ratio of existing channel cross-sectional area $(R_e)_i$ to the ultimate channel cross-sectional area $(R_e)_{ULT}$. An assessment was then carried out in order to determine the feasibility of providing storage and rate control within the existing pond. Ultimately, the erosion control benefit would be based on a combination of the difference between the existing channel cross-sectional area $(R_e)_i$, the ultimate channel cross-sectional area $(R_e)_{ULT}$ and the channel cross-sectional area $(R_e)_{CONT}$ using the optimal storage and rate control.

C. Potential Water Quality Benefit of Retrofitting Each Pond

Water quality control criteria selected was the Level 1 target **[Editor's Note: now referred to as enhanced protection level]**. A Level 1 target was selected because of the sensitivity of the receiving waters (the Rouge River and associated tributaries). Table 3.1 was then used to determine the required water quality storage volumes.

Step 4: Select SWM Practices Based on Evaluation Criteria

The eleven stormwater management ponds were initially evaluated against five technical criteria. Different priority values (weights) were given to each criteria.

- **Habitat index** – Higher habitat index value indicated a more sensitive stream and resulted in a higher priority for retrofitting the pond.

- **Ratio of catchment area draining to the pond (PCDA) and total catchment drainage area (CDA)** – Ponds with a high PCDA:CDA ratio were considered to be higher in priority for retrofit than those with a lower ratio since stormwater ponds that treat a higher percentage of the total catchment drainage area (CDA) are considered to have greater potential for protecting/restoring downstream erosion and water quality problems.

- **Ultimate stream area enlargement ratio** – Stormwater management ponds which drain to a receiver with a relatively high ultimate enlargement ratio were considered to be higher in priority for retrofit than ponds which discharge to a stream with a small ultimate enlargement ratio.

Table G.3: Summary of Existing Stormwater Management Ponds

Pond No.	Pond Name	Pond Type	Land Use (CDA)*	Land Use (PCDA)**
11.0	SE Quadrant Brown's Corner	On-Line/Dry Pond	11.9% Residential 31.1% Industrial 3.5% Airport 53.5% Undeveloped	20.0% Industrial 80.0% Undeveloped
12.0	Markville Pond	Off-Line/Wet Pond	8.0% Residential 5.9% Industrial 0.4% Airport 85.7% Undeveloped	60.0% Residential 40.0% Undeveloped
80.0	Leitchcroft Farm Pond 2	Off-Line/Dry Pond	14.6% Residential 62.0% Industrial 23.4% Undeveloped	100% Industrial
82.0	Beaver Creek Pond 1	On-Line/Dry Pond	56.2% Industrial 43.8% Undeveloped	n/a
82.1	Beaver Creek Pond 3	Off-Line/Dry Pond	13.6% Residential 33.2% Industrial 4.2% Airport 49.0% Undeveloped	100% Industrial
87.0	Hagerman Estates Subdivision	Off-Line/Dry Pond	7.5% Residential 89.6% Industrial 2.9% Undeveloped	100% Residential
88.0	Bridle Trail Phase 3	Off-Line/Dry Pond	15.2% Residential 84.8% Undeveloped	100% Residential
88.1	Bridle Trail Phase 4	Off-Line/Dry Pond	19.2% Residential 80.8% Undeveloped	80.0% Residential 20.0% Undeveloped
88.2	Bridle Trail Phase 5	Off-Line/Dry Pond	18.4% Residential 81.6% Undeveloped	90.0% Residential 10.0% Undeveloped
90.0	Raymerville Community	On-Line/Dry Pond	11.5% Residential 88.5% Undeveloped	100% Residential
98.0	Unionville B-3 Subdivision	Off-Line/Wet Pond	7.5% Residential 6.1% Industrial 0.4% Airport 86.0% Undeveloped	80.0% Residential 20.0% Undeveloped

* CDA = total catchment drainage area
** PCDA = pond catchment drainage area

Source: Best Management Practices Environmental Resource Management Project – Town of Markham, 1996.

- **Ratio of existing channel cross-sectional area to the ultimate channel cross-sectional area** – Stormwater ponds which drain to a receiving channel which has not yet reached an advanced stage of enlargement were considered to be higher in priority for retrofit than ponds which discharge to streams which have already undergone relatively significant enlargement.

- **Stream order** – Stormwater ponds which drain to a low order (smaller) receiver were considered to be higher in priority than those ponds which discharge to a higher (larger) receiver since low order streams are (all other factors being equal) more sensitive to a change in the flow regime.

Table G.4 summarizes the findings of the evaluation for each pond.

Subsequent evaluations focussed on the feasibility of retrofitting each pond, including the ability to expand storage volume, adjacent land uses, safety, access, etc.

Step 5: Develop an Implementation Plan

Implementation of the selected preferred option is the final step in the evaluation methodology but was not included in the Markham retrofit study. Implementation will involve prioritizing the eleven stormwater management ponds based on further technical evaluation criteria, feasibility, cost and other factors.

Table G.4: Evaluation of SWM Ponds for Retrofit Based on Technical Criteria
Town of Markham and T.R.C.A.

Pond No.	Pond Name	HI		PCDA/CDA		$(R_e)_{ult}$		$1 - \{(R_e)_i / (R_e)_{ult}\}$		SO		Total Weighted Score	Priority
		NPV	Rank	NPV	Rank	NPV	Rank	NPV	Rank	NPV	Rank		
11.0	SE Quadrant Brown's Corner	0.40	7.5	0.25	6	0.05	5	0.25	8	0.05	8.5	7.18	8
12.0	Waldon Pond (Markville)	0.40	5.5	0.25	10	0.05	8	0.25	5	0.05	10.5	6.88	7
80.0	Leitchcroft Farm Pond 2	0.40	10	0.25	2	0.05	2	0.25	2.5	0.05	4	5.43	4
82.0	Beaver Creek Pond 1	0.40	10	0.25	1	0.05	3	0.25	7	0.05	4	6.35	6
82.1	Beaver Creek Pond 3	0.40	7.5	0.25	9	0.05	4	0.25	9	0.05	4	7.90	11
87.0	Hagerman Estates Subdivision	0.40	10	0.25	4	0.05	1	0.25	10	0.05	4	7.75	10
88.0	Bridle Trail Phase 3	0.40	2.5	0.25	8	0.05	10	0.25	2.5	0.05	4	4.33	3
88.1	Bridle Trail Phase 4	0.40	2.5	0.25	7	0.05	6	0.25	2.5	0.05	4	3.88	2
88.2	Bridle Trail Phase 5	0.40	2.5	0.25	3	0.05	7	0.25	2.5	0.05	4	2.93	1
90.0	Raymerville Community	0.40	2.5	0.25	5	0.05	11	0.25	11	0.05	8.5	5.98	5
98.0	Unionville B-3 Subdivision	0.40	5.5	0.25	11	0.05	9	0.25	6	0.05	10.5	7.43	9

Notes:

1. NPV – Normalized Priority Value.
2. HI – Habitat Index.
3. PCDA – Catchment Drainage Area draining to pond.
4. CDA – total Catchment Drainage Area.
5. $(R_e)_{ult}$ – Ultimate Stream Area Enlargement Ratio.
6. $(R_e)_i$ – Existing Stream Area Enlargement Ratio.
7. SO – Stream Order.
8. Rank and Priority Scale – 11 (low priority) to 1 (high priority).

Source: Best Management Practices Environmental Resource Management Project – Town of Markham, 1996.

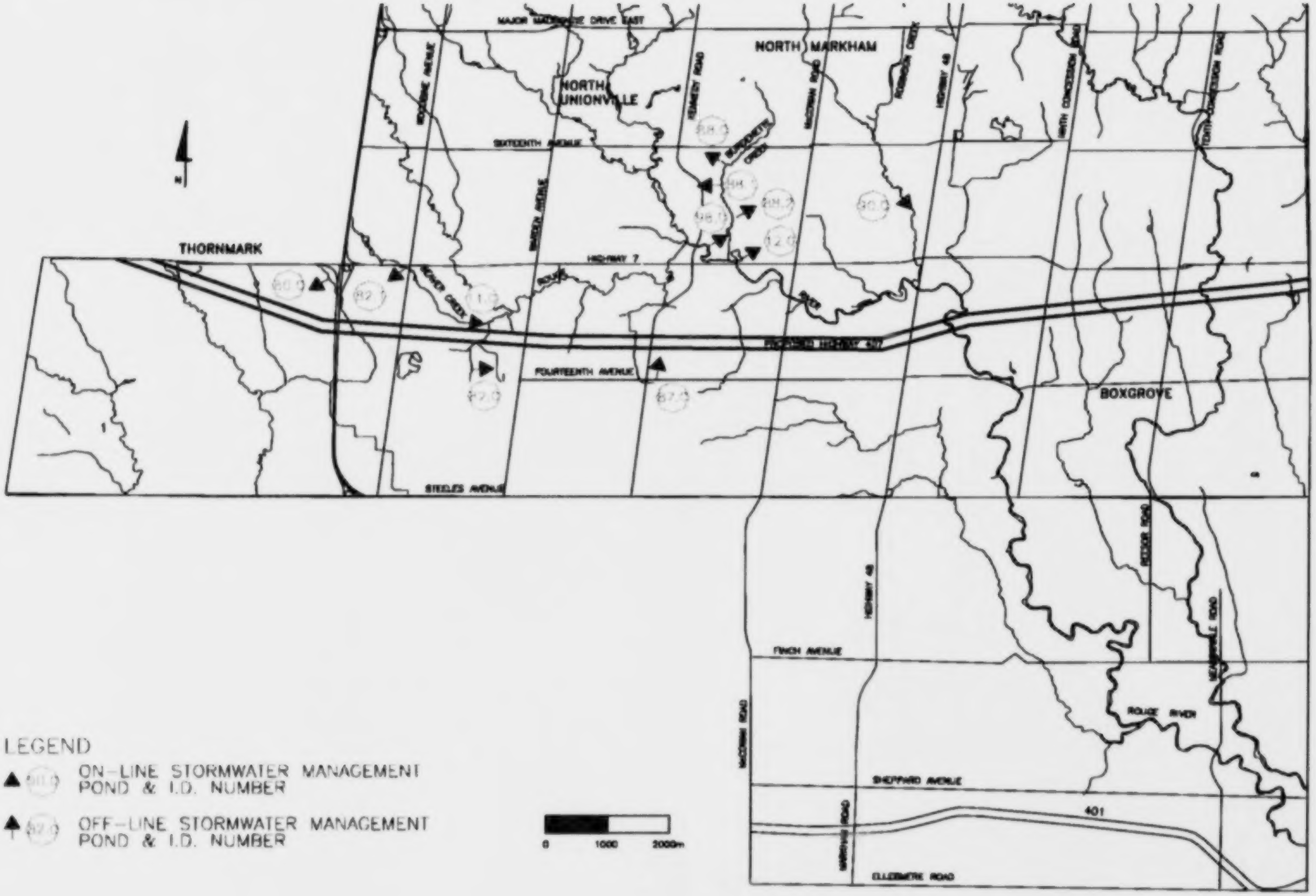

MAJOR MACKENZIE DRIVE EAST
NORTH MARKHAM
NORTH UNIONVILLE
THORNMARK
BOXGROVE
WOODBINE AVENUE
WARDEN AVENUE
KENNEDY ROAD
MCCOWAN ROAD
ROBINSON CREEK
HIGHWAY 48
NINTH CONCESSION ROAD
TENTH CONCESSION ROAD
SIXTEENTH AVENUE
BRUCE CREEK
HIGHWAY 7
ROUGE RIVER
PROPOSED HIGHWAY 407
FOURTEENTH AVENUE
STEELES AVENUE
REESOR ROAD
MEADOWVALE ROAD
ROUGE RIVER
FINCH AVENUE
SHEPPARD AVENUE
401
MARKHAM ROAD
ELLESMERE ROAD
80.0
82.1
11.0
87.0
87.0
88.0
98.0
88.2
12.0
90.0
LEGEND
ON-LINE STORMWATER MANAGEMENT POND & I.D. NUMBER
OFF-LINE STORMWATER MANAGEMENT POND & I.D. NUMBER
0
1000
2000m
N

Figure G.1: Location of Stormwater Management Ponds within Study Area – Markham, Ontario

Source: The Toronto and Region Conservation Authority and the Town of Markham, Town of Markham Stormwater Retrofit Study, 1999.

APPENDIX H
SWMP SAMPLE CALCULATIONS*

** Note: The examples are based on the Stormwater Management Practices Planning and Design Manual (1994).*

H.1 Case I: Development is Governed by a Subwatershed Plan

The proposed development is within an area which has a subwatershed plan with the following stormwater management criteria:

- quantity control to reduce the 1 in 5 year post-development peak flow to pre-development levels;
- quality control to detain the runoff volume from a 25 mm rainfall event for 24 hours;
- erosion control equivalent to 100 m^3/ha to be detained for 24 hours; and
- baseflow maintenance of 10 mm/ha based on soils with a percolation rate of 70 mm/h.

The proposed development site is 4.5 ha and will consist of 100 townhouses with a total imperviousness of 63%. The soils in the area have an average percolation rate of 50 mm/h.

Based on the subwatershed plan, the total developed area will require 450 m^3 for erosion control (storage for 24 hours). Using OTTHYMO (Wisner and P'ng, 1983), the runoff volume was modelled for the total site for a 4 hour 25 mm rainfall event, and it was determined that approximately 566 m^3 is required for water quality control. Therefore, stormwater management controls are required to detain 566 m^3 for 24 hours to address water quality and erosion control criteria.

H.1.1 Lot Level Controls

Reduced Lot Grading

Based on the soils and the type of development, the lot grades will be reduced from 2% to 0.5%. Since the land is naturally flat, reduced lot grading will be feasible. The lots will be graded at 2% within 4 m of the building and at 0.5% for the remainder of the lot.

$$DSP = 4.67 + (2 - G)\,f$$

Equation 4.13: Adjusted Pervious Depression Storage

where G = 0.5% (lot grading)
f = 0.75 (longevity factor)

Using Equation 4.13, the pervious depression storage (DSP) was adjusted based on the longevity factor. The adjusted DSP used in the model was 5.8 mm to account for the reduced lot grading.

Roof Leader Discharge to Soakaway Pits

Since residential rooftop drainage is considered "clean water," the roof leaders from the buildings will be discharged to rear yard soakaway pits. The trenches will be located approximately 4 m away from the buildings and approximately 1.5 m above the seasonally high water table. They will be filled with 50 mm diameter clear stone and each trench will be lined with non-woven filter cloth to prevent clogging of the stone. The appropriate bottom area of each trench was calculated using Equation 4.3. Each soakaway pit will serve four townhouse units; therefore, each trench will need to be able to store a maximum volume of 20 mm over the rooftop area of four units (approximately 400 m^2). For the 100 units, there will be a total of 25 trenches.

$$A = \frac{1{,}000V}{Pn\Delta t}$$

Equation 4.3: Infiltration Trench Bottom Area

where V = 8 m^3 (runoff volume to be infiltrated: 20 mm × 400 m^2 rooftop area for four units)
P = 50 mm/h (percolation rate of surrounding native soil)
n = 0.4 (porosity for clear stone)
Δt = 24 h (retention time)

In order to infiltrate this amount of water, the trench bottom area (A) needs to be at least 16.7 m^2. Based on the lot configuration and open space areas, soakaway pits which are 2 m wide and 8.5 m long can be constructed. For the storage volume of 8 m^3, the pit needs to be 1.2 m deep.

Based on Equation 4.2, the maximum allowable soakaway pit depth is 1.2 m deep.

$$\text{Maximum Allowable Soakaway Pit depth} = P\,T$$

Equation 4.2

where P = 50 mm/h (minimum percolation rate)
T = 24 h (drawdown time)

The required pit depth of 1.2 m (for 8 m^3 storage volume) is within the range of maximum allowable soakaway pit depth (Equation 4.2).

Equation 4.17 was used to calculate a rating curve for input to the model based on the storage and outflow for all the soakaway pits:

$$Q = f \times \left(\frac{P}{3{,}600{,}000}\right) \times (2LD + 2WD + LW) \times n$$

Equation 4.17: Soakaway Pit Rating Curve

$$V = LWD \times n \times f$$

where f = 0.75 (longevity factor)
P = 50 mm/h (native soil percolation rate)
L = 212.5 m (total length of the soakaway pits)
D = 1.2 m (depth of water in the soakaway pit)
W = 2 m (width of each soakaway pit)
n = 0.4 (void space in the soakaway pit clear stone)

Therefore, for a volume of 153 m^3, the discharge will be 0.004 m^3/s. This rating curve was modelled using OTTHYMO and the ROUTE RESERVOIR command for a 4 hour 25 mm storm to assess the contribution of the soakaway pit storage in the determination of end-of-pipe water quality storage requirements. Overflows from the trench storage were added to the runoff from the rest of the site.

H.1.2 Conveyance Controls

Pervious Pipe Systems

The townhouse development will be serviced with traditional curb and gutters. Groundwater contamination is not an issue for this development since a shallow aquifer feeds the stream and the road is local and will not be salted or sanded. Therefore, pervious pipes will be used with regular storm sewers for overflows. The municipality's standards allow pervious storm sewer systems. Grassed boulevards will be used as pre-treatment for the stormwater runoff. A total length of 260 m (130 m on each side of the roads) of perforated pipe with fifty 12.7 mm diameter perforations per metre will be used. The 200 mm diameter perforated pipe will be set at 0.5% slope to promote exfiltration. Clear stone (50 mm) will be used for pipe bedding. The bedding will be surrounded with non-woven filter fabric to prevent the native soil from clogging the voids. The maximum depth will be 1.2 m as calculated previously using Equation 4.1. A typical pervious pipe design is shown in Figure 4.11 (Chapter 4).

Based on the following equation, a rating curve was estimated for the perforated pipe exfiltration flow as a percentage of the pipe flow.

$$Q_{exf} = (15A - 0.06S + 0.33)\,Q_{inf}$$

Equation 4.18: Exfiltration Discharge

where Q_{exf} = exfiltration flow through pipe perforations (see Table H.1)
A = 0.006 m^2/m (area of perforations/m length of pipe)
S = 0.5% (slope of pipe)
Q_{inf} = flow in pipe (see Table H.1)
f = 1.0 (longevity factor)

Table H.1: Head Versus Exfiltration Flow for Perforated Pipe

Depth of water in pipe (m)	Flow in Pipe (m^3/s)	Exfiltration Flow (m^3/s)
0	0	0
0.025	0.001	0.0004
0.05	0.003	0.001
0.075	0.0065	0.003
0.1	0.012	0.005
0.125	0.0165	0.007
0.15	0.021	0.008
0.175	0.022	0.009
0.2	0.023	0.009

The following equation was used to determine the amount of storage volume available within the clear stone pipe bedding.

$$V = LWD \times n \times f$$

where L = 260 m (length of pervious pipe and stone)
W = 3.0 m (width of stone)
D = 1.2 m (depth of stone)
n = 0.4 (void space for clear stone)
f = 0.75 (longevity factor based on native soil)

Therefore, the actual available volume (V) within the storage media is 281 m^3. The COMPUTE DUHYD command in OTTHYMO was used to divert the peak exfiltration flow to the pipe bedding. The exfiltrated flow was routed through the storage volume using the ROUTE RESERVOIR command.

The outflow from the pipe bedding (soakaway pit rating curve) was calculated based on Equation 4.17.

$$Q = f \times \left(\frac{P}{3{,}600{,}000}\right) \times (2LD + 2WD + LW) \times n$$

Equation 4.17: Soakaway Pit Rating Curve

where f = 0.75 (longevity factor)
P = 50 mm/h (native soil percolation rate)
L = 260 m (total length of the soakaway pits)
D = 1.2 m (depth of water in the soakaway pit)
W = 3.0 m (width of each soakaway pit)
n = 0.4 (void space in the soakaway pit clear stone)

The outflow from the pipe bedding is 0.006 m^3/s. All overflows were separated from the exfiltrated flows once the pipe bedding storage was exceeded. The overflows were conveyed to the regular storm sewer and used to determine end-of-pipe stormwater management requirements.

Based on the OTTHYMO output, the entire pipe bedding storage is not required. Therefore, as a cost-saving measure, the storage volume was reduced to 140 m^3 (width was reduced to 1.5 m and the corresponding outflow was 0.004 m^3/s). **Note:** An alternative approach would have been to increase the number of perforations and hence, the exfiltration in the perforated pipe.

H.1.3 End-of-Pipe SWMPs

Quality Control

According to the runoff volume reported in the OTTHYMO modelling, the required end-of-pipe storage is 275 m^3. The contributing drainage area and runoff volume are too small for the design of a wet pond or wetland. Therefore, a sand filter is recommended to provide the remaining water quality control. Based on the area available for the sand filter, Equation 4.20 was used to calculate the outflow from the sand filter.

$$Q = f \times \left(\frac{P}{3{,}600{,}000} \right) \times (LW \times n)$$

Equation 4.20: Sand Filter Discharge

where f = 1.0 (longevity factor based on the percolation rate for sand)
P = 210 mm/h (percolation rate for sand)
L = 32 m (length of the filter)
W = 8 m (width of the filter)
n = 0.25 (void space in the sand filter)

Therefore, the outflow from the filter will be 0.004 m^3/s. The storage available within the sand filter is 32 m^3. Storage to a depth of 1.0 m above the sand filter will be used to provide 256 m^3 of active storage. The ROUTE RESERVOIR command was used to model the storage and outflow rating curve.

To provide control of the 1 in 5 year post-development peak flow, a dry pond is recommended which will receive 1 in 5 year flows from the storm sewers. The pond will provide 520 m^3 of storage at approximately 1.0 m depth. The outlet was sized to control the 1 in 5 year post-development peak flow to the pre-development flow.

H.1.4 Baseflow

The reported percolation rate of the soil is actually 50 mm/ha. Therefore, using Equation H.1, the actual infiltration target is 7 mm/ha.

$$I = V \left(\frac{P_{site}}{P_{SWP}} \right)$$

Equation H.1: Site-Specific Infiltration Adjustment

where V = 10 mm/ha (target volume of infiltration from subwatershed plan based on a specific storm event)
P_{site} = 50 mm/h (percolation rate of site-specific soils)
P_{SWP} = 70 mm/h (percolation rate of soils used in subwatershed plan)

Based on the infiltration measures recommended for this site, the total amount of recharge is 14.73 mm/ha which is greater than the required 7 mm/ha to meet the adjusted infiltration target.

H.1.5 Summary of Case I

Based on the stormwater management criteria outlined in the subwatershed plan for this site, quantity control, quality control, erosion control and baseflow maintenance are required. The following stormwater management design will meet each of these criteria.

i) the 1 in 5 year post-development peak flow will be controlled with a dry pond approximately 520 m^3 in volume;

ii) the reduced lot level grading and soakaway pits will reduce the required water quality storage by storing 15 mm (based on the longevity factor) of runoff from the roof area (approximately 150 m^3);

iii) the pervious pipe system will further reduce the water quality storage by providing storage in the pipe bedding (approximately 140 m^3);

iv) the sand filter will provide the remaining water quality storage (approximately 275 m^3);

v) the stormwater management controls will double the required baseflow contribution (approximately 14 mm/ha); and

vi) the measures designed for water quality control will also provide erosion control benefits.

H.2 Case II: No Subwatershed Plan Governs Development

In the absence of watershed/subwatershed planning, Chapter 3 of the SWMP manual was used to provide guidance on the design of stormwater management controls for a 50 ha subdivision. The proposed level of imperviousness for the site is 55%. The entire development will consist of 950 single detached housing units on typical 12 m × 30 m lots. Since there are no flood damage sites downstream of the site, and the site is located at the downstream end of the watershed, the site does not require flood control. The level of protection for aquatic habitat for the receiving water course is normal protection.

H.2.1 Lot Level Controls

Based on the soils, the potential for use of lot level controls is low. The soils have a percolation rate of 20 mm/h, and within this municipality, flat lot grading (< 2%) is not permitted. Also, the potential for contamination of the groundwater is a concern. Therefore, the only lot level control recommended for this site is soakaway pits for rooftop drainage.

Roof Leader Discharge to Soakaway Pits

Since rooftop drainage is considered "clean water," the roof leaders from the buildings will be discharged to rear yard soakaway pits. The trenches will be located approximately 4 m away from the buildings and approximately 1.5 m above the seasonally high water table. They will be filled with 50 mm diameter clear stone. Each trench will be lined with non-woven filter cloth to prevent clogging of the stone.

According to Table 4.11, the water quality storage requirements for the site should be reduced based on the use of soakaway pits. The appropriate bottom area of each trench was calculated using Equation 4.3. Each rooftop is approximately 102 m^2. Equation 4.3 was used to calculate the bottom area required to store the maximum volume of 20 mm over the rooftop area.

$$A = \frac{1{,}000V}{Pn\Delta t}$$

Equation 4.3: Infiltration Trench Bottom Area

where V = 2.04 m^3 (runoff volume to be infiltrated for 1 lot)
P = 20 mm/h (percolation rate of surrounding native soil)
n = 0.4 (porosity for clear stone)
Δt = 24 h (retention time)

Therefore, the bottom area of each trench would have to be 10.6 m^2. An area of 5.4 m^2 can be accommodated on each lot (1.2 m wide and 4.5 m long). Based on Equation 4.2, the maximum allowable soakaway pit depth is as follows:

$$\text{Maximum Allowable Soakaway Pit depth} = PT$$

Equation 4.2

where P = 20 mm/h (minimum percolation rate)
T = 24 h (drawdown time)

The maximum soakaway pit depth is 0.5 m. Based on the maximum depth and bottom area which can be accommodated, 10 mm of roof drainage can be accommodated in the soakaway pits.

A total of 1,026 m^3 storage will be provided in soakaway pits for the subdivision (950 lots).

H.2.2 Conveyance Controls

Traditional curb and gutters will service this development. Based on the infiltration rates of the soils on this site and the potential for groundwater contamination, pervious pipes are not recommended.

H.2.3 End-of-Pipe SWMPs

A wet pond was chosen as the end-of-pipe stormwater management facility for this subdivision. According to Table 3.2, the design of a wet pond will require 110 m^3/ha of storage which corresponds to the following storage volumes for 50 ha: 3,500 m^3 for permanent pool and 2,000 m^3 for extended detention storage. The wet pond will be located outside of the floodplain and will have a length-to-width ratio of 4:1. The permanent pool will be 2 m deep, and the extended detention storage will be approximately 1.25 m deep.

Storage Requirements

Equation 4.16 determines the reduction in the required end-of-pipe water quality storage volume (active storage) as given by Table 3.2, based on the use of soakaway pits for rooftop drainage.

$$V = [(A - RS) \times S] + [(RS \times S) - (SPV \times f)]$$

Equation 4.16: Water Quality Storage Volume Required

where V = volume of water quality storage required (m^3)
A = 50 ha (total area of site)
RS = 9.69 ha (total roof area for all 950 lots)
S = 110 m^3/ha (water quality storage requirement from Table 3.2)
f = 0.5 (longevity factor)

and SPV = $LWD \times n$ (volume of soakaway pit storage)

where L = 4,275 m (length of all soakaway pits)
W = 1.2 m (width of each soakaway pit)
D = 0.5 m (depth of each soakaway pit)
n = 0.4 (void space in the soakaway pit clear stone)

Therefore, the required end-of-pipe active water quality storage volume is reduced from 2,000 m^3 to 1,487 m^3.

Temperature

Since the receiving water course is sensitive to temperature changes, Equation H.2 was used to calculate the temperature change in the stream. Equation H.3 was used to calculate the average urban runoff temperature.

$$\Delta T_{stream} = \left(\frac{QT + q(T_{urb} + \Delta T_{SWMP})}{(Q + q)} \right) - T$$

Equation H.2: Temperature Mass Balance

where Q = 0.233 m^3/s (average monthly summer daily maximum flow rate in the stream)
T = 20°C (average monthly summer temperature in the stream)
T_{urb} = 20.2 (average urban runoff summer temperature)
q = 0.03 m^3/s (average flow from SWMP during a 15 mm storm event)
ΔT_{SWMP} = 5.1°C (average increase in temperature by SWMP type (Table 4.3))

$$T_{urb} = 15.8 + 0.08\,(55)$$

Equation H.3: Urban Runoff Temperature

Therefore, the change in stream temperature (ΔT_{stream}) is 0.60°C.

Erosion

There are a variety of methods that designers can use to determine appropriate erosion control requirements including the Simplified Design Approach and the Detailed Design Approach (see Chapter 3 – Environmental Design Criteria and Appendices B, C and D).

A subwatershed study was not performed for this site. The following example outlines a method that has been used by the Toronto and Region Conservation Authority (TRCA) and assumes that required erosion control would be 24 hour detention for a 25 mm rainfall event.

The required volume is 6,875 m^3 which is greater than the 1,487 m^3 required for water quality control (Table 3.2). The required volume for the pond will be decreased by the soakaway pit volume for a total required volume of 6,362 m^3 (6,875 m^3 – 513 m^3 provided by the soakaway pits).

The soils in the area are clayey silts and silty clays. Therefore, according to Figure 4.6, the critical velocity for a 0.01 mm size of particle is approximately 45 cm/s or 0.45 m/s. OTTHYMO was then used to model the erosion control volume to determine if the critical velocity is surpassed in the downstream channel. The uncontrolled post-development flows exceed the critical velocity resulting in an index value of 625.25 based on Equation H.4.

$$E_I = \sum (V_i - V_c)\Delta t \qquad \textbf{Equation H.4: Erosion Index}$$

where E_I = erosion index
V_i = 1.18 m/s (velocity in the channel at time $t = 1.5$ h ($> V_c$))
0.72 m/s (velocity in the channel at time $t = 1.667$ h ($> V_c$))
0.49 m/s (velocity in the channel at time $t = 1.834$ h ($> V_c$))
V_c = 0.45 m/s (critical velocity above which erosion will occur)
Δt = 601.2 s (timestep (0.167 h))

Flows under pre-development and controlled post-development conditions do not exceed the critical velocity. Therefore, the 25 mm control is adequate for this site.

Drawdown Time

The drawdown time in the pond can be estimated using Equation 4.10.

$$t = \frac{2A_p}{CA_o(2g)^{0.5}}\left(h_1^{0.5} - h_2^{0.5}\right) \qquad \text{Equation 4.10: Drawdown Time}$$

or if a relationship between A_p and h is known (i.e., $A = C_2h + C_3$)

$$t = \frac{0.66 C_2 h^{1.5} + 2 C_3 h^{0.5}}{2.75 A_o} \qquad \text{Equation 4.11}$$

where A_p = varies (surface area of the pond)
C = 0.62 (discharge coefficient)
A_o = 0.04 m^2 (cross-sectional area of the orifice for 226 mm diameter)
g = 9.81 m/s^2 (gravitational acceleration constant)
h_1 = varies (starting water elevation above the orifice)
h_2 = varies (ending water elevation above the orifice)
C_2 = 4371 (slope coefficient from the area-depth linear regression)
h = 1.09 m (maximum water elevation above the centre-line of orifice)
C_3 = 3220 (intercept from the area-depth linear regression)

The linear regression was based on the area versus depth (y) listed.

for: A_p = 3,136 m^2 $h_1 = 0$ m
A_p = 3,969 m^2 $h_1 = 0.14$ m (0.25 - 0.113)
A_p = 4,900 m^2 $h_1 = 0.39$ m (0.5 - 0.11)
A_p = 5,929 m^2 $h_1 = 0.64$ m (0.75 - 0.11)
A_p = 7,056 m^2 $h_1 = 0.89$ m (1 - 0.11)
A_p = 8,036 m^2 $h_1 = 1.09$ m (1.2 - 0.11)

$$A_p = 4{,}371\,h + 3{,}220$$

$$A_o = \frac{3{,}282 + 6{,}724}{2.75t} \qquad \text{(from Equation 4.10)}$$

$$t = \frac{3{,}639}{A_o}$$

Therefore, the drawdown time in the pond is equal to 89,752 s or 24.9 hours.

Forebay Length

The forebay size depends on several calculations.

1. Settling Calculations

The first step is to determine the distance to settle out a certain size of sediment in the forebay. The settling velocities for different sized particles can be estimated from the stormwater particle size distribution monitoring data by the U.S. EPA. Equation 4.5 defines the appropriate forebay length for a given settling velocity.

$$\text{Dist} = \sqrt{\frac{rQ_p}{V_s}}$$

Equation 4.5: Forebay Settling Length

where r = 2:1 (length-to-width ratio of forebay)
Q_p = 0.1 m^3/s (peak flow rate from the pond during design quality storm)
V_s = 0.0003 m/s (settling velocity for 0.15 mm diameter particles)

Therefore, the forebay should be 26 m long to settle particles approximately 0.15 mm diameter in size.

2. Dispersion Length

Equation 4.6 provides a simple guideline for the length of dispersion required to dissipate flows from the inlet pipe. It is recommended that the forebay length is such that a fluid jet will disperse to a velocity ≤ 0.5 metre/second at the forebay berm. The fluid jet should be based on the capacity of the inflow pipe (if the pipe is ≤ 10 year pipe). In this subdivision, the pipe will be designed to convey the 5 year storm flows. A flow splitter will not be implemented.

$$\text{Dist} = \frac{8Q}{dV_f}$$

Equation 4.6: Dispersion Length

where Q = 5.1 m^3/s (inlet flow rate)
d = 2 m (depth of the permanent pool in the forebay)
V_f = 0.5 m/s (desired velocity in the forebay)

Therefore, the forebay length should be 40.8 m for the peak flow during a 5 year storm.

A guideline for the minimum bottom width of this deep zone is given by:

$$\text{Width} = \frac{\text{Dist}}{8}$$

Equation 4.7: Minimum Forebay Bottom Width

Therefore, the forebay deep zone should be at least 5.1 m wide.

Therefore, the forebay will be 45 m long and 20 m wide (based on an approximate 2:1 length-to-width ratio). The velocity of the flow as it moves through the forebay will be as follows:

$$\text{Velocity} = \frac{Q}{A}$$

where Q = 5.1 m^3/s
A ≈ 22 m^2 (cross-sectional area)

Therefore, the average velocity through the forebay will be 0.23 m/s. This velocity is acceptable since it is less than the 0.45 m/s permissible velocity to prevent erosion, as noted previously.

Given the results of Equations 4.5 and 4.6, the forebay length will be 45 m long and 20 m wide. The permanent pool volume of the forebay will be approximately 900 m^3.

3. Clean-out Frequency

Based on Table 7.3, the annual sediment loading for this site will be approximately 2,300 kg/ha or 1.9 m^3/ha. Therefore, based on the volume of the forebay (900 m^3) and a pond removal efficiency of 70% (Level 2 protection **[Editor's Note: now referred to as normal level of protection]**), the forebay will be required to be cleaned out every 13.5 years. This is acceptable to the municipality since it is greater than its 0 year minimum cleanout frequency.

Forebay Berm

The forebay will be separated from the rest of the pond by an earthen berm. The berm will be submerged slightly below the permanent pool. Low flow pipes will be installed in the berm to convey low flows from the forebay to the pond. The conveyance pipes will be installed in the berm at 0.6 m above the bottom of the forebay. A maintenance pipe will also be installed in the berm to drawdown the forebay for maintenance purposes.

H.2.4 Summary of Case II

According to Table 3.1, a wet pond for this site will require 3,500 m^3 for a permanent pool and 2,000 m^3 for active storage to provide water quality control. For erosion control, the required volume is 6,875 m^3 based on the 25 mm rainfall event. The following SWMPs have been designed to meet these criteria:

i) Soakaway pits will accommodate 10 mm of runoff from the roof area which will reduce the required end-of-pipe active storage requirements by 513 m^3; and

ii) A wet pond will provide the end-of-pipe stormwater management (water quality and erosion) control. The pond will provide 3,500 m^3 of permanent pool storage and 6,362 m^3 of active storage.

APPENDIX I
STORMWATER MANAGEMENT PRACTICES – DESIGN EXAMPLES*

** Note: Examples showcase real world projects that were developed in the 1990s and are based on the 1994 Manual. The 2003 Manual has updated many concepts. Therefore, designers must refer to the 2003 Manual to ensure that future projects are designed in accordance with current standards.*

I.1 Introduction

Users of the *Stormwater Management Practices Planning and Design Manual* (1994) indicated that design examples would be useful to show the level of detail required in stormwater design submissions and supporting documents for applications to approval agencies. The examples provided in this Appendix are typical of submissions made in the 1990s in various Ontario municipalities. The following points should be noted when reviewing the design examples:

i) While the majority of the requirements are similar across different geographical locations, there may be standards specific to individual municipalities and districts. Designers should obtain specific municipal and other approval agency guidelines in order to incorporate these requirements into their design and obtain the necessary approvals.

ii) The *Stormwater Planning and Design Manual* promotes an integrated planning and design process based on the "treatment train" approach to the control of stormwater (lot level controls, conveyance controls, and end-of-pipe stormwater management facilities). Lot level controls (e.g., flatter grading of rear yards to promote infiltration) are generally incorporated in the overall grading design for the development. Generally, there are no specific drawings or details submitted for approval beyond the usual detailed grading plans. This is not to downplay the importance of lot level controls, but simply reflects the way in which they are normally incorporated in the stormwater management design.

iii) The SWMP facility should be designed so that it is an integrated component of the area serviced. For example, overland flow routes must be carefully designed to ensure that flows reach end-of-pipe facilities that provide major system flood control. Similarly, the effect of peak water levels in an end-of-pipe facility on the hydraulic grade line in the storm sewers must be carefully considered to avoid surcharging and possible basement flooding problems.

iv) At the detailed design stage, drawings for stormwater management facilities are frequently submitted for approval as part of an overall subdivision design package including: grading plans, drainage plans, detailed plans and profiles of storm and sanitary sewers, water mains, other utilities, road profiles, etc. This appendix does not contain examples of all these drawings. Also, in order to avoid duplication, only selected items from the complete design package have been included to illustrate a specific type of SWMP. A complete submission in such cases would be more extensive than shown.

I.2 Design Example 1 – End-of-Pipe Extended Detention Facility (Quantity and Quality Control)

A two-cell facility which separates water quality and erosion control from quantity control will be discussed in this example. However, single-cell facilities for all types of control are more commonly used. Single-cell ponds are similar to Design Example 2 (see Section I.3) with quantity control storage being provided above the erosion control storage level.

The facility is located within a new primarily single-family residential community and provides quantity and quality control for 66 hectares of storm runoff (Figure I.1). The quality control cell was designed as an artificial wetland, and the quantity control cell was designed as a dry detention area to receive flows only when the quality pond filled.

The "Stormwater Servicing Plan" (SSP) (essentially a simplified subwatershed plan) design criteria for the facility were developed in consultation with the Town, the Conservation Authority and the District Office of the Ministry of Natural Resources. Approval was obtained from the Ministry of the Environment through the delegated authority of the Region.

The SSP design criteria were:

Flood Control
Post-development peak flows to be controlled to pre-development levels for the lands draining to the facility for 2 to 100 year design storm events. In addition, supplementary flood control storage was incorporated to ensure peak flows further downstream in the subwatershed remained at pre-development levels.

Erosion Control
Twenty-four hour detention for the runoff from a 40 mm storm was incorporated.

Water Quality
Storage was based on the 1994 SWMP Manual requirements for Level 1 protection **[Editor's Note: now referred to as enhanced protection]** including 40 m^3/ha of active storage. This active storage was in addition to that provided for flood and erosion control.

The following design drawings were included and are illustrated in this chapter:

- Plan view (Figure I.1) at a scale of 1:500 (reduced copy of the plan – Figure I.2);
- Example of detail sheet showing the design of inlet and outlet structures (Figure I.3); and
- Two detailed planting plans at a scale of 1:500 showing the design of the artificial wetland and plantings around the border of the facility (Figures I.4 and I.5, respectively).

These drawings were accompanied by a "Stormwater Management Report" for the community which updated information contained in the "Stormwater Servicing Plan" and included a description of the functional design of the facility referred to as the South Pond. It is somewhat more extensive than a design brief which would typically accompany the design drawings. A more typical example is included in association with Design Example 2 (Section I.3).

Figure I.1: Plan View

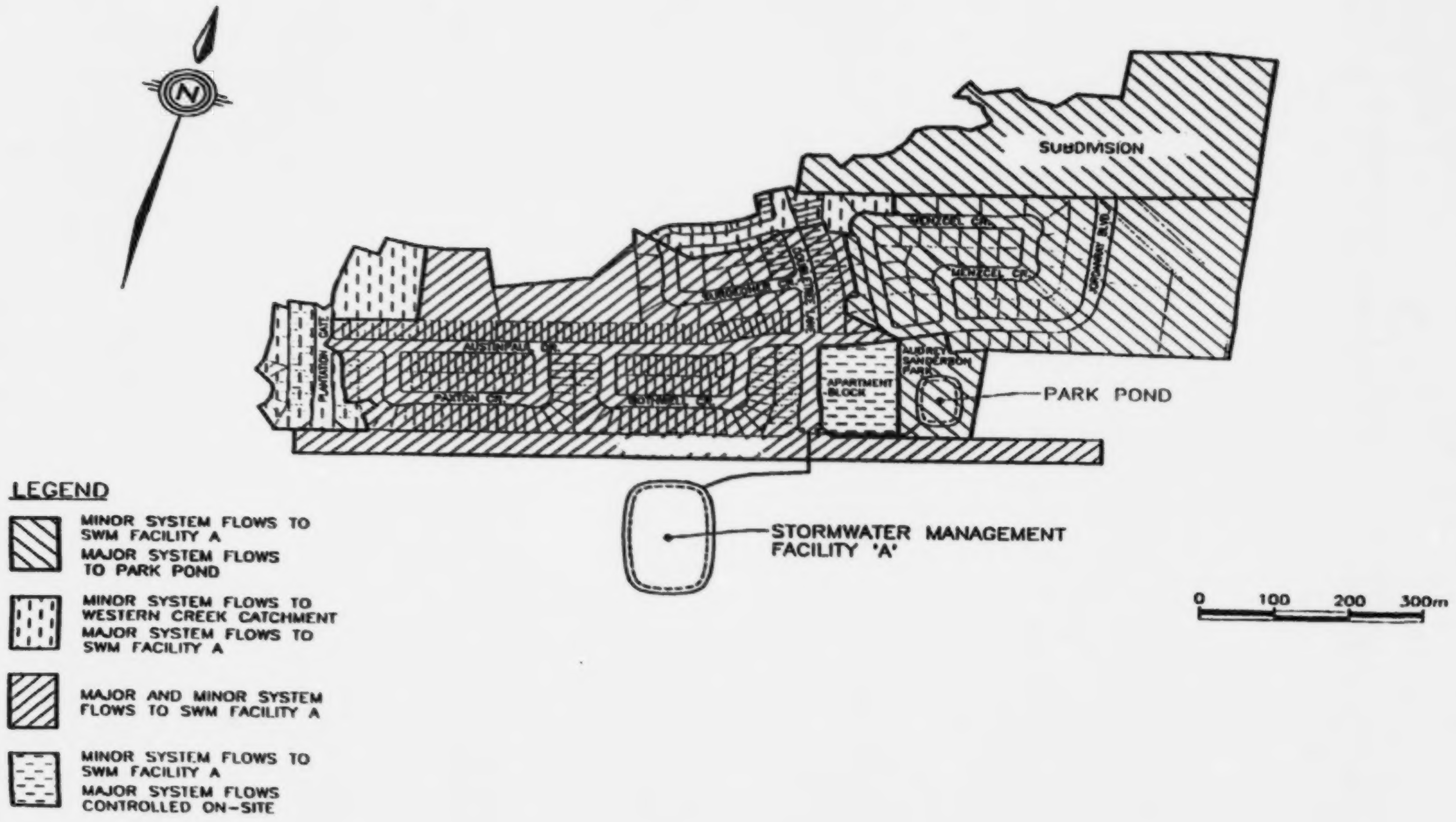

N
SUBDIVISION
MENZEL CR.
MENZEL CR.
JORDANRAY BLVD.
COUNTRYSIDE LANE
PLANTATION GATE
AUSTINPAUL CR.
PAXTON CR.
BOTHWELL CR.
APARTMENT BLOCK
AUDREY SANDERSON PARK
PARK POND
STORMWATER MANAGEMENT FACILITY 'A'
LEGEND
MINOR SYSTEM FLOWS TO SWM FACILITY A
MAJOR SYSTEM FLOWS TO PARK POND
MINOR SYSTEM FLOWS TO WESTERN CREEK CATCHMENT
MAJOR SYSTEM FLOWS TO SWM FACILITY A
MAJOR AND MINOR SYSTEM FLOWS TO SWM FACILITY A
MINOR SYSTEM FLOWS TO SWM FACILITY A
MAJOR SYSTEM FLOWS CONTROLLED ON-SITE
0
100
200
300m

Figure I.2: Plan View (reduced copy)

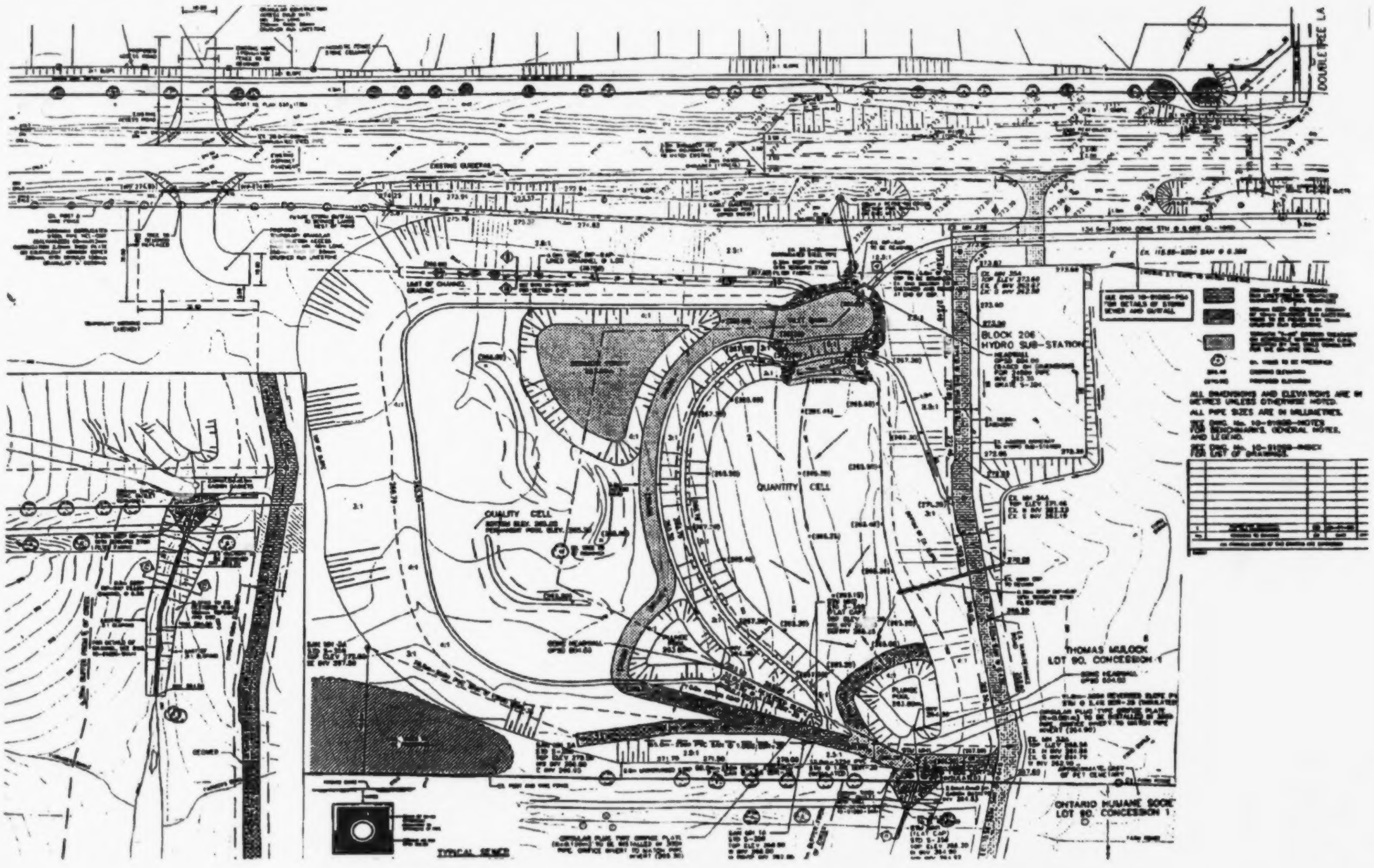

DOUBLE TREE LA
BLOCK 206
HYDRO SUB-STATION
QUANTITY CELL
QUALITY CELL
THOMAS MULOCK
LOT 90, CONCESSION 1
ONTARIO HUMANE SOCIE
LOT 90, CONCESSION 1
TYPICAL SEWER
ALL DIMENSIONS AND ELEVATIONS ARE IN METRES UNLESS OTHERWISE NOTED.
ALL PIPE SIZES ARE IN MILLIMETRES.

Figure I.3: Detail Sheet showing the Design of Inlet and Outlet Structures

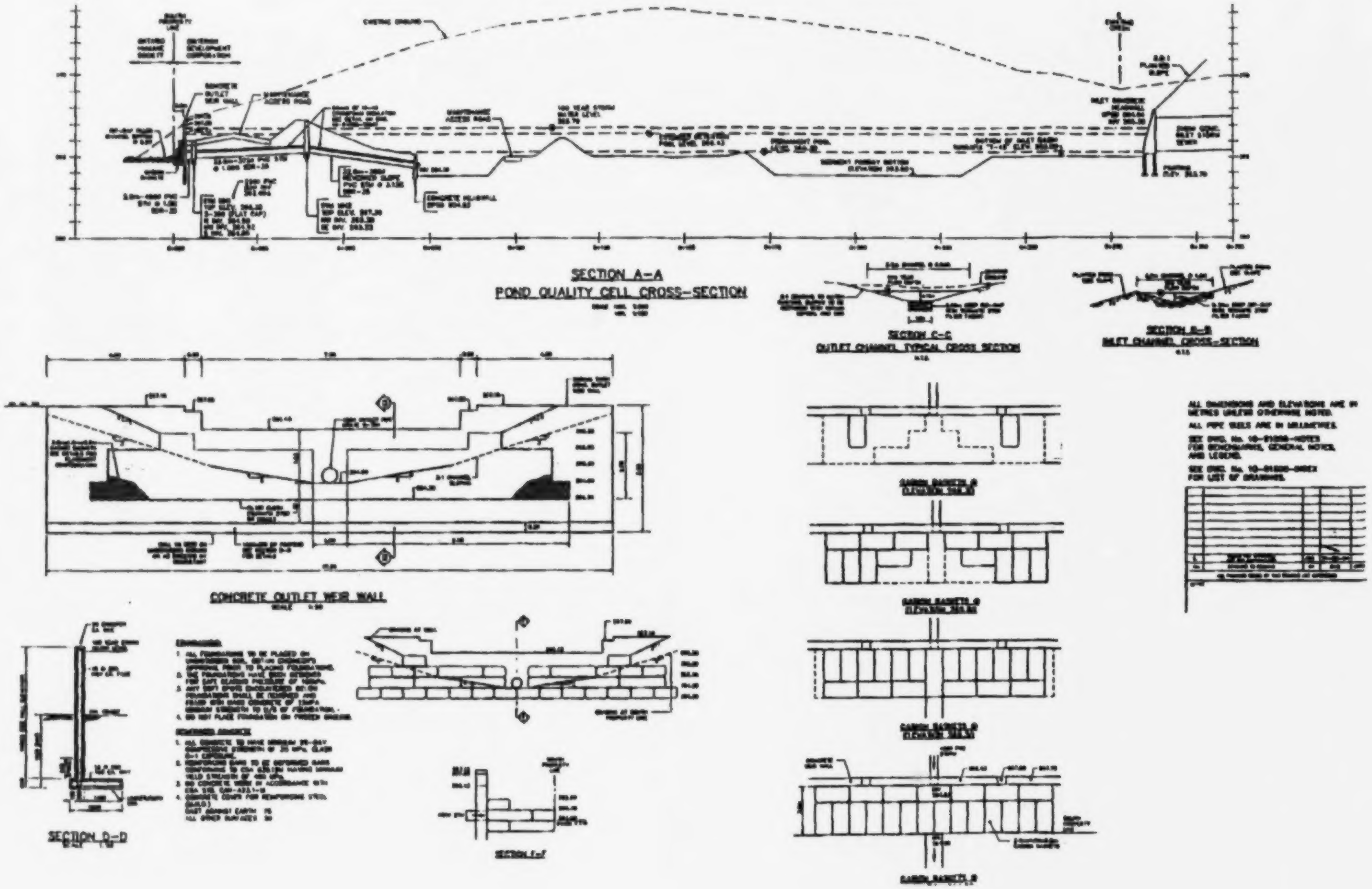

SECTION A-A
POND QUALITY CELL CROSS-SECTION
SECTION C-C
OUTLET CHANNEL TYPICAL CROSS SECTION
SECTION B-B
INLET CHANNEL CROSS-SECTION
CONCRETE OUTLET WEIR WALL
SECTION D-D
ALL DIMENSIONS AND ELEVATIONS ARE IN METRES UNLESS OTHERWISE NOTED.
ALL PIPE SIZES ARE IN MILLIMETRES.

Figure I.4: Planting Plan showing the Design of the Artificial Wetland

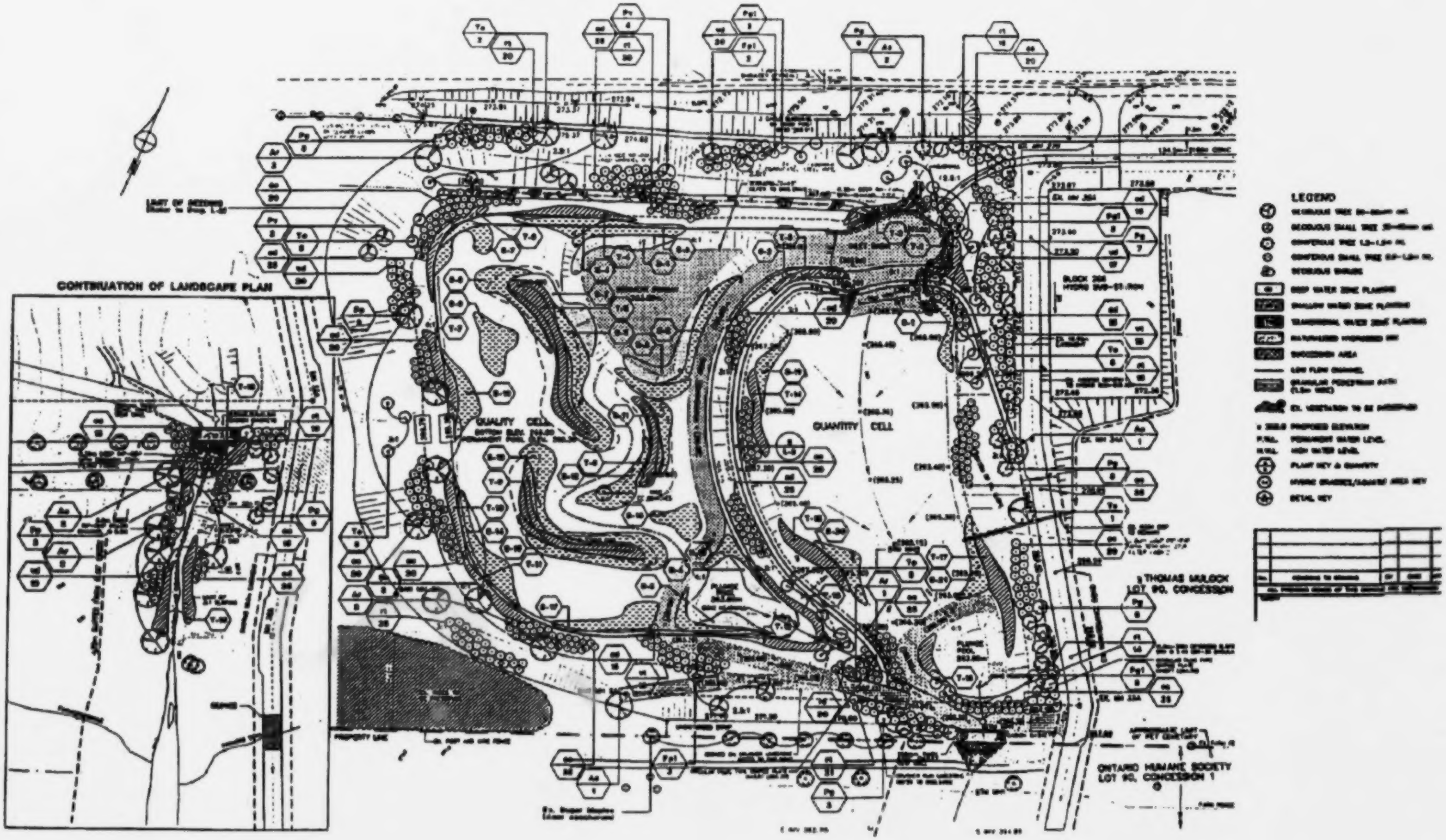
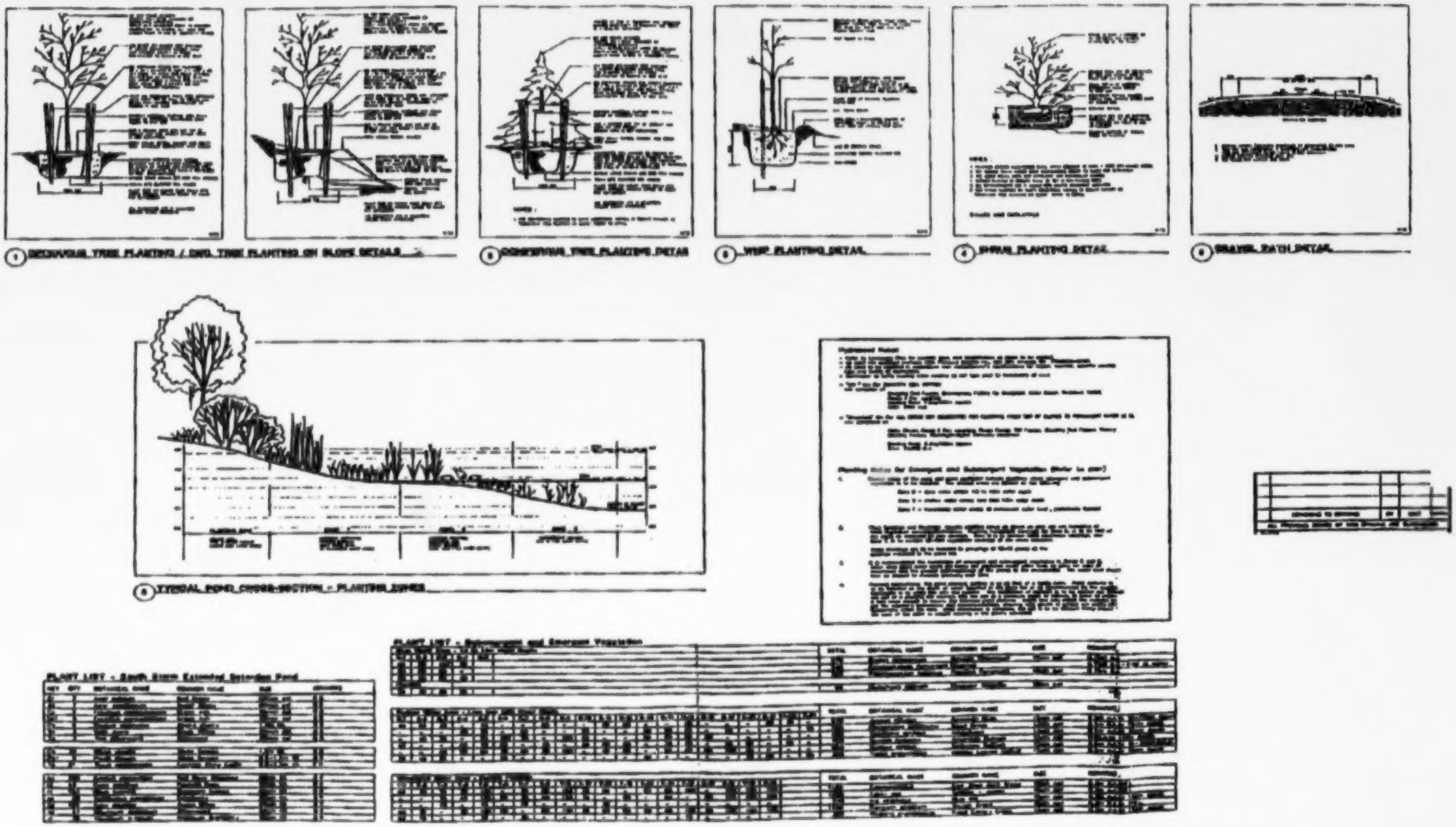

CONTINUATION OF LANDSCAPE PLAN
QUALITY CELL
QUANTITY CELL
LEGEND
THOMAS MULOCK
LOT 90, CONCESSION
ONTARIO HUMANE SOCIETY
LOT 90, CONCESSION 1

Figure I.5: Planting Plan showing Plantings around the Border of the Facility



I.3 Design Example 2 – End-of-Pipe Extended Detention Facility (Quality and Erosion Control Only)

The single-cell facility discussed in this section was designed to provide water quality and erosion control. The design can be extended to include quantity control by providing additional storage above the erosion storage.

The facility is located within a new retirement residential community (detached homes on relatively small lots) surrounded by a 9 hole golf course. The stormwater pond provides water quality control for storm runoff for about 10 hectares and was designed to be part of the golf course. The pond was designed to implement one of the recommendations of the "Stormwater Management (SWM) Plan." SWM Plan design criteria were developed in consultation with the Town, the Conservation Authority, and the District Office of the Ministry of Natural Resources. Approval was also obtained from the Ministry of Environment and Energy.

The design criteria for the SWM Plan were:

Flood Control
Since the pond drains directly into a sizeable lake, there was no requirement to control post-development peak flows to pre-development levels.

Erosion Control
Twenty-four hour detention for the runoff from a 25 mm storm was incorporated.

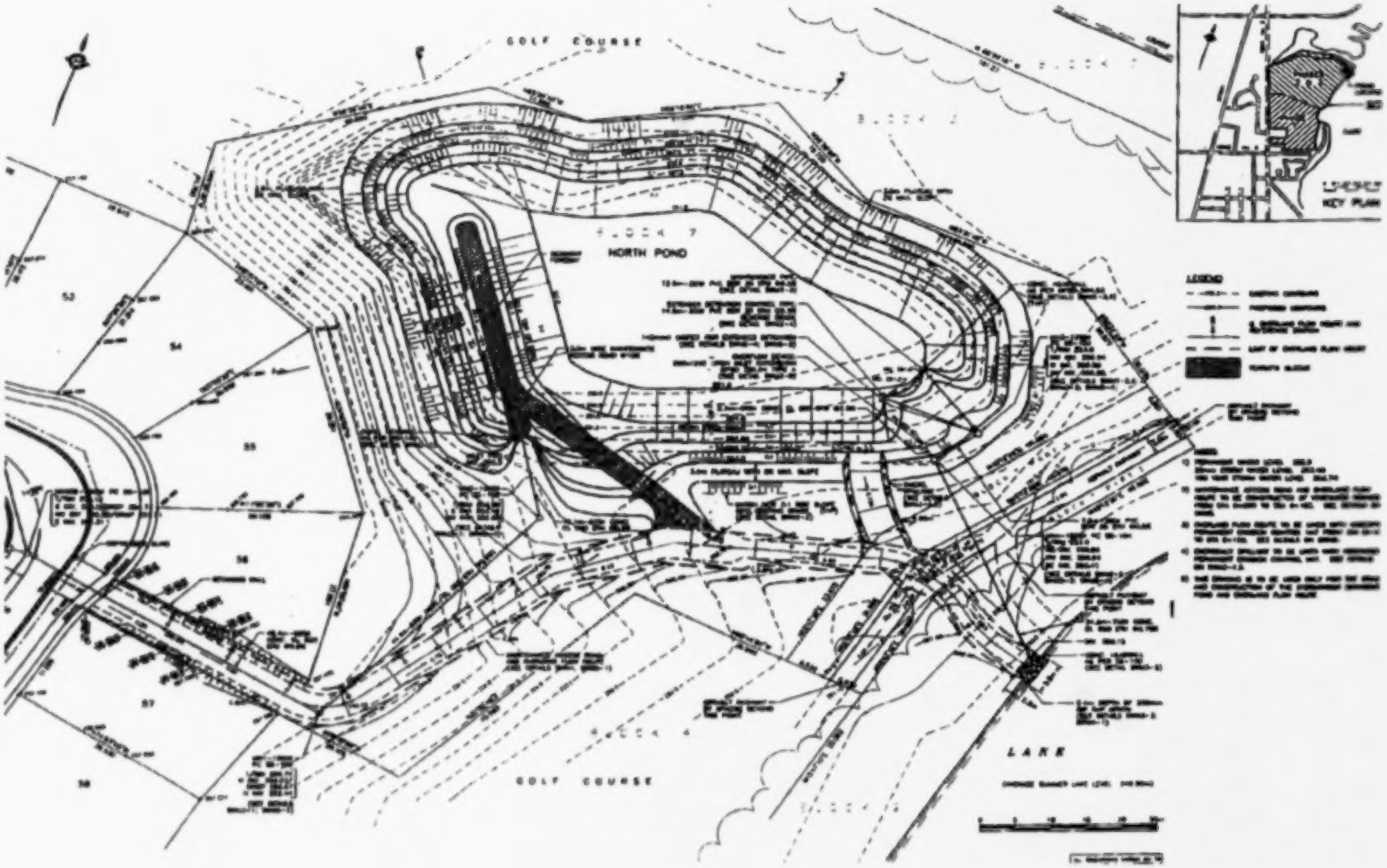
Water Quality
Permanent pool storage based upon the 1994 SWMP Manual requirements for Level 1 protection was incorporated **[Editor's Note: now referred to as enhanced protection]**. The active storage requirement of 40 m^3/ha for water quality control was taken as part of the 25 mm (erosion) detention storage.

The facility was designed as a single-cell extended detention pond with wetland plantings incorporated in certain areas. The inflow to the pond is restricted to 5 year flows from the storm sewer system. The major system flows are diverted around the facility via an overland flow route directly to the lake.

The design drawings included:

- Plan view at a scale of 1:500 **[Editor's Note: not included]** (reduced copy of the plan – Figure I.6);
- Four detail sheets showing the design of inlet/outlet structures and overland flow routes **[Editor's Note: not included]**; and
- Detailed planting plans prepared by a landscape consultant showing the wetland plantings around the border of the facility **[Editor's Note: not included]**.

Figure I.6: Plan View (reduced copy)


GOLF COURSE
BLOCK 7
NORTH POND
LEGEND
NOTES
KEY PLAN
LAKE
BLOCK 4
GOLF COURSE

These drawings were accompanied by a "Functional Design Report" describing the design criteria, final hydrologic modelling of the facility, storage calculations, inlet and outlet design, maintenance and access features, and the overland flow route design. Excerpts from this report are provided in the following sections to indicate the level of detail typically included in such a document.

I.3.1 Functional Design Report Example

Introduction
Proponent X is developing a residential subdivision and golf course in Township X. It is located on the shores of Lake XXXX, east of Road Number 2 and north of Road Number 8. As illustrated in Figure 1, the development is scheduled to proceed in three phases **[Editor's Note: Figure 1 has not been included]**. Construction of Phase 1 and its associated services has commenced. The primary stormwater management practice that has been implemented for this portion of the development is an extended detention quality control facility located adjacent to Lake X. This facility, a wet pond, has been named the South Water Quality Pond. It is proposed that an extended detention wet pond for quality control also be implemented to service both Phases 2 and 3 of the development and has been named the North Water Quality Pond. The purpose of this report is to describe the detailed design of the North Pond.

Background
The justification and general design criteria for the proposed stormwater management facility are provided in the "Stormwater Management Study," 199x. Given the quality concerns in the receiving Lake, the Study recommended that the facility consist of an extended detention wet pond sized to provide quality control of runoff for the areas tributary to it. In accordance with the 1991 MOEE/MNR Interim Stormwater Quality Control Guidelines for New Developments, the active storage requirements for quality control in the pond were based on detaining the runoff from a 25 mm storm for 24 hours. Current guidelines as given in the Stormwater Management Practices Planning and Design Manual (MOEE, 1994) recommend only 40 m^3 of extended detention per hectare of area draining to the facility. This more recent guideline tends to generate much smaller storage requirements for extended detention than the 1991 guideline. However, given the sensitive nature of the receiving Lake, the former, more conservative criteria has been retained for the purposes of determining the active storage requirements for erosion quality control in the North Pond. Permanent pool requirements for quality control will be based on the guidelines given in the 1994 MOEE SWMP Manual **[Editor's Note: designers should refer to the 2003 SWM Planning and Design Manual for guidelines]**.

As specified in the Stormwater Management Study, all minor system flow from the areas tributary to the proposed North Pond will be directed to the facility. The minor system has been designed to convey the five year event. Therefore, the peak flow that will be conveyed to the facility under the 5 to 100 year storms is the 5 year post-development flow from the area draining to it. Major system flow generated in this area will be conveyed to the Lake by means of an overland flow route designed to convey major system flows generated up to the 100 year storm.

It should be noted that prior to the submission of the Stormwater Management Study, the estimated drainage area for the North Pond was 12.34 hectares. Given the current development scheme and grading plan, the drainage area will instead be 9.25 hectares as shown in Figure X **[Editor's Note: not included]**. The average percent imperviousness of this area will be approximately 45%.

Approval Requirements
It is anticipated that this report and the accompanying drawings will provide the required documentation for the following approvals:

- Township (Plan Approvals);
- Ministry of Natural Resources (Work Permit for Storm Outfall to Lake);
- Ministry of Environment and Energy (Certificate of Approval); and
- Conservation Authority (Plan Approvals and Fill and Construction Permit).

Design Criteria
The Lake has been classified as a Class 1 habitat requiring Level 1 protection **[Editor's Note: now referred to as enhanced protection]**. In order to ensure the protection and enhancement of the Lake and its watershed, a series of design criteria were identified for the North Pond. Criteria were established in the "Stormwater Management Study" as well as taken from the detailed design phase of the South Pond and are described below.

Functional Criteria
The following criteria must be satisfied to ensure that the water quality control requirements are met:

i) The facility should be designed as a wet pond. The minimum permanent pool volume in the facility should be 125 m^3/ha of area draining to the facility. This volume is the minimum recommended permanent pool volume in a wet pond facility designed to provide Level 1 treatment for a 45% impervious area as specified in the 1994 MOEE SWMP Manual **[Editor's Note: now referred to as enhanced protection; designers should refer to the 2003 SWM Planning and Design Manual]**. While there is the potential to create a permanent pool with a volume larger than the minimum MOEE recommended volume, consideration must be given to the permanent pool volume that can actually be sustained by the contributing drainage area. Typically, it is desirable to have a 30 day turnover in the permanent pool. The maximum permanent pool volume should be determined by taking into consideration the historical average monthly rainfall depths in the vicinity of the site and the monthly runoff expected given the imperviousness of the site and typical rainfall depth distributions for southern Ontario as specified in the MOEE SWMP Manual.

ii) The facility should have sufficient active storage for 24 hour detention of the runoff from the contributing drainage area under a 25 mm event. This criteria was established to satisfy the quality concerns for the receiving Lake.

iii) Storm runoff from the area tributary to the pond will be conveyed to the facility by means of a minor system designed to convey the five year event. The facility should be designed with sufficient active storage to pass the peak minor system flow without overflowing.

iv) The facility should be designed with an emergency overflow weir.

v) Major system flow from the subdivision will be conveyed to the Lake by means of an overland flow route. According to the Township design criteria for open channels, the maximum flow velocity in the overland flow route should be 2.5 m/s.

vi) Any ponding that occurs at the low point on the road adjacent to the pond (the major overland flow route) must not extend beyond the curb line except at the location of the entrance to the overland flow route.

Environmental, Aesthetic and Safety Criteria
The following criteria must be met to ensure that the facility provides environmental benefits, is attractively integrated into its surroundings, and presents a minimum hazard to the public:

i) The maximum permanent pool depth should be 2.0 m. The maximum active pool depth should be 1.5 m.

ii) A minimum length-to-width ratio of 3:1 should be maintained in the pond ensuring the pollutant removal benefits associated with a longer flow path.

iii) The facility should be designed with a sediment forebay to improve pollutant removal by trapping larger particles near the inlet of the pond. The forebay should be 1-2 m deep to minimize the potential for re-suspension and to prevent the conveyance of re-suspended material to the pond outlet. The forebay dimensions should be selected to provide maximum dispersion of the inflow to the pond, thereby reducing velocities in the cell.

iv) Side slopes around the facility should vary to present a natural appearance. Terraced grading should be used to discourage public access to the pond.

v) The storm outfall to the Lake should be designed to create a minimum of disturbance within the 15 m buffer around the Lake.

vi) The stormwater management block (Block 7) that will contain the proposed facility will be bordered on the north and south by a golf course. A golf cart path should extend across the eastern end of the stormwater management block connecting the paths at the northern and southern limits of the block. An easement has been created for this path.

vii) The permanent water level in the pond should be such that the pond creates a visual amenity to the golf course.

Maintenance and Access Criteria

i) A hard surface should be installed in the forebay of the quality cell. The hard surface should be capable of withstanding the weight of the small grading equipment that will be used to periodically clean the forebay **[Editor's Note: this practice is no longer recommended in the 2003 SWM Planning and Design Manual – see Chapter 4]**.

ii) An access road should be provided to and from the forebay. The outlet structure from the pond and the pond outfall to the Lake should also be made accessible.

iii) The extended detention control device should be located within an easily accessible manhole rather than within the wetted area of the pond (i.e., perforated risers should not be used).

iv) A maintenance pipe should be provided to permit draining of the permanent pool.

Hydrologic Modelling Approach

In order to determine the active storage requirements in the North Pond for quality control and the design flows for the overland flow route, the hydrologic models (OTTHYMO) described in the "Stormwater Management Study" were retrieved and updated to reflect the current drainage scheme for the Phases 2 and 3 lands. The 9.25 hectares that will drain to the North pond were modelled as one basin using the STANDHYD command. Total and directly connected impervious values used in the model were 45% and 30%, respectively. A characteristic slope of 2% was used to reflect the proposed lot and street grading on the subject lands. A curve number of 78 was used based on the silty sand soils on the site and the proposed land use. A DUHYD command was used to split the minor and major system flows. Minor system flows were taken to be the 5 year flow generated on the site. The model was run with the 4 hour 5, 25 and 100 year Chicago distribution storms, as well as a 2 hour 25 mm storm. A simulation was also conducted with a 4 hour 25 mm storm. However, since the runoff volumes generated under this storm were smaller than the shorter duration 25 mm storm, the 2 hour 25 mm storm was taken to be critical.

Modelling Results

The minor system flows and 5 through 100 year major system flows derived from the OTTHYMO simulations are summarized in Table I.1. The minor system flows are comparable to those derived using the Rational method as illustrated in the Table. Under the 25 mm storm, the runoff depth from the proposed development is 12.22 mm. This translates into a runoff volume of 1,130 m^3. The detailed modelling results are included in the appendix **[Editor's Note: not included]**. The storm sewer design sheets for the areas draining to the pond are given in the appendix **[Editor's Note: not included]**.

Table I.1: Summary of OTTHYMO Results

Minor System (5 Year) Flows m^3/s		Major System Flows m^3/s		
OTTHYMO	**Rational Method**	**OTTHYMO 5 Year**	**OTTHYMO 25 Year**	**OTTHYMO 100 Year**
1.07	0.94	0	0.86	1.68

Functional Design of North Pond and Overland Flow Route

The following sections describe the detailed design of the North Pond and illustrates how each of the design criteria will be met.

North Pond Storage Requirements

Permanent Pool

Using the MOEE SWMP Manual guidelines, the minimum required permanent pool volume for the proposed facility is 1,160 m^3 **[Editor's Note: designers should refer to the 2003 Manual]**. The maximum permanent pool volume was determined by multiplying the historical average monthly rainfall depth near the site in the driest month of the summer by a weighted runoff coefficient.

This coefficient was derived by recognizing that the amount of runoff generated under a given event will depend on the depth of the storm and by assuming that the frequency distribution of rainfall depth near the site is the same as the frequency distribution reported for typical southern Ontario sites in the MOEE SWMP Manual. A review of the historical average monthly precipitation records for the closest climatological station (with the same approximate elevation as the site) indicates that September is the summer month with the smallest average rainfall. The rainfall in this month is 65.1 mm. Using a runoff coefficient of 0.38 for the subject property, the maximum permanent pool volume is approximately 2,300 m^3.

Drawing X shows the grading proposed to provide the required storage **[Editor's Note: not included]**. As can be seen in the drawing, the facility has a 1 m deep permanent pool and the permanent water level is 252.0 m. The permanent pool volume is approximately 1,980 m^3.

Active Pool

The results of the OTTHYMO modelling indicate that the required active pool volume for erosion control is 1,130 m^3. As shown in Table I.2, this volume is provided just below an active depth of 0.40 m (elevation 252.40 m). Water ponding up to this depth will drain by means of a reverse sloped pipe fitted with an orifice plate sized for 24 hour drawdown. Any water ponding above the 252.40 elevation will drain by means of a ditch inlet catchbasin located at the southeast corner of the pond. If the water elevation in the pond should reach an elevation of 252.75, the emergency weir will begin to operate.

Table I.2: Elevation – Active Storage Relationships for North Pond

Elevation (m)	Active Storage (m^3)
252.00	0
252.20	580
252.40	1,200
252.60	1,870
252.80	2,570
253.00	3,340

As stated earlier, storm runoff from the area tributary to the pond will be conveyed to the facility by means of a minor system pipe designed to convey the five year event. Since the facility will accept 5 year post-development flows from its contributing drainage area, it is important that the facility have sufficient active storage to pass the 5 year flow without surcharging. In order to confirm that no surcharging will occur, two OTTHYMO simulations were conducted:

1. to predict the maximum water elevation in the pond under a 100 year storm assuming that there is no active storage in the pond at the start of the storm and that the ditch inlet catchbasin and orifice are fully operational; and

2. to predict the maximum water elevation in the pond under a 100 year storm assuming that the water level in the pond is 252.40 m at the start of the storm and that the ditch inlet catchbasin and the orifice are blocked (i.e., the only outlet from the pond is the emergency overflow weir).

The simulations were conducted by performing a ROUTE RESERVOIR command on the minor system hydrograph calculated using the DUHYD command. The storage outflow tables used in the ROUTE RESERVOIR command were tailored to reflect the different scenarios. The results of these simulations are illustrated in Table I.3 which indicates that under normal operating conditions (i.e., outlets not blocked), the maximum water level in the pond will be 252.74 m. Under extreme operating conditions, the maximum water elevation in the pond will be 252.94 m. The detailed results of these simulations are included in Appendix X **[Editor's Note: not included]**.

Table I.3 Maximum Water Elevations – North Pond

Run 1: Water level is 252.00 m at start of storm. All outlets working			
	5 Year Storm	25 Year Storm	100 Year Storm
Peak Outflow From Pond (m^3/s)	0.17	0.30	0.44
Maximum Water Elevation (m)	252.56	252.65	252.74
Run 2: Water level is 252.35 metres at start of storm. Ditch Inlet Catchbasin (DICB) and Orifice Blocked			
	5 Year Storm	25 Year Storm	100 Year Storm
Peak Outflow From Pond (m^3/s)	0.29	0.49	0.73
Maximum Water Elevation (m)	252.86	252.90	252.94

North Pond Inlet Design

The design (5 year peak) inflow into the pond is 1.07 m^3/s. Careful consideration has been given to the design of the pond inlet in order to minimize the potential for re-suspension of previously settled material in the wet pond. The proposed inlet design is described below:

i) An 825 mm diameter pipe will convey the minor system flow from manhole 209 on the adjacent road to the pond. The invert of the pipe at the pond inlet is 252.0 m.

ii) There is a 1 m deep sediment forebay at the entrance to the pond. This pond is separated from the rest of the pond by a 0.8 m berm. The length and width of the forebay have been sized according to the dispersion, volume and surface area criteria given in the 1994 MOEE SWMP Manual **[Editor's Note: designers should refer to the 2003 SWM Planning and Design Manual]**.

iii) The inlet will be protected from erosion by placing erosion control blocks along the erosion-prone areas adjacent to the pipe.

North Pond Outlet Design

In designing the outlet for the North Pond, the following was considered:

- providing 24 hour drawdown of the volume required for extended detention;
- passing flows in excess of the first flush out of the pond without causing surcharging; and
- providing easy access for maintenance.

The outlet from the pond is described below:

i) The extended detention control device for the pond will consist of a 300 mm diameter reverse sloped pipe fitted with a 142 mm diameter orifice at its connection to MH 3 (see Drawing X **[Editor's Note: not included]**). The orifice has been sized to provide 24 hour detention of the 1,200 m^3 of active storage available below an elevation of 252.40 m. The invert of the orifice will be set at an elevation of 252.0 m. The calculations used for sizing the orifice are included in Appendix X **[Editor's Note: not included]**.

ii) A 1,200 mm × 600 mm Type A ditch inlet catchbasin will convey runoff in excess of the first flush out of the pond. The invert of the ditch inlet catchbasin will be 252.40 m. The ditch inlet catchbasin drains to a 600 mm diameter pipe connected to MH 4. A 750 mm diameter pipe will extend from this manhole to the Lake.

iii) A 4.6 m wide emergency overflow weir set at an elevation of 252.75 m will convey flows out of the facility in the event that the other outlets are not functioning properly. The weir has been sized to pass a flow equivalent to the design flow into the facility (i.e., 1.07 m^3/s) with a maximum water elevation in the pond of 253.0 m. The emergency spillway will convey flows to the overland flow route into the Lake. The emergency spillway will be protected from erosion by means of a vegetated erosion control mat. Assuming a maximum water elevation in the pond of 253.0 m, the maximum velocities on the emergency spillway will be 1.93 m/s.

The stage-storage-outflow relationships for the pond are given in Appendix X **[Editor's Note: not included]**.

Maintenance and Access Features for North Pond

Specific attention has been give to the maintenance and access features of the facility. In particular, the following items should be noted:

i) A 4 m wide access route constructed of a vegetated, perforated cellular confinement system backfilled with 30/70 topsoil/sand mix will extend from the adjacent road to the forebay of the pond. The cellular confinement system has been designed to provide load support for the small grading equipment that will occasionally be required to clean the forebay.

ii) The forebay itself will be lined with erosion control blocks which have been sized to support the maintenance equipment that will periodically be required for sediment removal **[Editor's Note: this practice is no longer recommended in the 2003 Manual]**.

iii) A 300 mm diameter maintenance pipe will be provided at the pond outlet to facilitate draining the pond. The pipe will be fitted with a gate valve as shown in Drawing 1 **[Editor's Note: not included]**.

iv) The slide frame that will contain the orifice gate at the outlet of the extended detention control pipe has been designed such that it can also be used to isolate the extended detention control pipe if required.

v) The manholes at the pond outlet will be accessible from the asphalt pathway shown in Drawing 1 **[Editor's Note: not included]**. A second pathway (to be constructed by others) will extend to the pond outfall at the Lake.

Design of Overland Flow Route

A major system outlet for the Phases 2 and 3 lands will be created between Lots 56 and 57 on the adjacent road. An overland flow route will be constructed to convey major system flows from the low point on the road, easterly between lots 56 and 57 and then through the stormwater management block towards the Lake. The average slope for the route will be approximately 6.1%. The route will be constructed as described below:

i) The first 103 m of the overland flow route will double as the maintenance access road for the pond. This portion of the route will be constructed of a vegetated, perforated cellular confinement system backfilled with 30/70 topsoil/sand mix. Given the 100 year design flow of 1.68 m^3/s, the maximum velocity on this portion of the route will be 2.24 m/s. The velocity calculations for the overland flow route are included in the appendix **[Editor's Note: not included]**.

ii) The final 47 m of the route will be lined with a vegetated, permanent erosion control mat. Given the 100 year design flow of 1.69 m^3/s (overland flow plus flow from the emergency spillway), the maximum velocity on this portion of the overland flow route will be 2.28 m/s.

iii) The overland flow route will pass over the asphalt pathway located in Part 2 of Block 7. Given the 100 year design flow of 1.69 m^3/s, the maximum flow depth and velocity on the pathway will be 0.16 m and 2.28 m/s, respectively.

iv) At the outlet of the overland flow route, a 2 m long, 5 m wide rip-rap apron will be constructed to protect the shoreline from erosion. The rip-rap will have a median diameter of 200 mm and will be placed to a depth of 400 mm. The erosion control mat that will line the overland flow route will extend underneath the rip-rap to prevent any native fines from being washed away.

v) The maximum ponding elevation at the low point on the adjacent road will be 259.30 m. The ponding limits on the road will not extend beyond the curb line except at the location of the entrance to the overland flow route and at several of the driveways located near the low point. The ponding limits are shown in Figure E-2 in the appendix. The ponding calculations are also given in the appendix **[Editor's Note: not included]**.

Summary

1. The Lake has been classified as a Level 1 habitat. Quality control of stormwater runoff for the proposed Phases 2 and 3 of the subdivision is required to meet guidelines for protecting a Level 1 habitat.

2. Quality control for the proposed development will be provided in an extended detention wet pond.

3. The permanent pool in the wet pond will have a volume of 1,980 m^3. This volume is greater than the 1,160 m^3 volume recommended in the 1994 SWMP Manual for wet ponds that are to provide Level 1 quality control **[Editor's Note: now referred to as enhanced protection in the 2003 SWM Planning and Design Manual]** of runoff from a 45% impervious area. Given the historical rainfall records in the area, it is predicted that the permanent pool will have an approximate 30 day turnover during the driest months of the summer.

4. An extended detention control device at the outlet of the wet pond will provide 24 hour drawdown of the 1,130 m^3 that will runoff from the proposed development under a 25 mm event.

5. Storm runoff from the area tributary to the pond will be conveyed to the facility by means of a minor system designed to convey the five year event. A ditch inlet catchbasin at the outlet of the pond will convey flows in excess of the first flush out of the pond. These flows will be conveyed to the Lake by means of a 750 mm diameter storm sewer.

6. An emergency overflow weir will convey flows out of the pond in the event that the ditch inlet catchbasin and/or orifice become blocked.

7. A 300 mm diameter maintenance pipe has been provided at the pond outlet in the event that the permanent pool needs to be drained.

8. The pond has been designed with a 1 m deep sediment forebay. The forebay will provide benefits with respect to settling and dispersion.

9. A 4 m wide access route constructed of a vegetated, perforated backfilled cellular confinement system with 30/70 topsoil/sand mix will extend from the adjacent road to the forebay of the pond. The cellular confinement system has been designed to provide load support for the small grading equipment that will occasionally be required to clean the forebay. The forebay itself will be lined with erosion control blocks **[Editor's Note: this practice is no longer recommended in 2003 Manual]**.

10. The major system outlet for the subdivision will be between Lots 56 and 57 on the adjacent road. An overland flow route will be constructed to convey major system flows from the low point on the road, easterly between lots 56 and 57 and then through the stormwater management block towards the Lake. The first 103 m of the overland flow route will double as the maintenance access road for the pond. This portion of the route will be constructed of a vegetated, perforated cellular confinement system backfilled with 30/70 topsoil/sand mix. The final 47 m of the route will be lined with a vegetated, permanent erosion control mat. The 100 year flow velocities on all portions of the route will be below the 2.5 m/s maximum specified in the Township criteria for design of open channels.

11. The maximum (100 year) ponding elevation at the low point on the adjacent road near the entrance to the overland flow route will be 259.30 m. The ponding limits on the road do not extend beyond the curb line except at the location of the entrance to the overland flow route and at several of the driveways located near the low point.

I.4 Example 3 – Integrated "Treatment Train" Infiltration System (Quality and Quantity Control)

This example is located within a new 28.3 hectare residential development. This development forms the first part of a larger 250 hectare subwatershed that is being developed based on the criteria contained within the design report **[Editor's Note: not included]**. A "treatment train" approach was used to design the lot level, conveyance and end-of pipe stormwater management facilities required.

The receiving outlet is a Level I coldwater stream within a Provincially Significant Wetland **[Editor's Note: requires what is now referred to as enhanced protection]**. In the undeveloped state, there is little or no surface runoff discharged to the receiving stream. Under normal rainfall events (2 to 100 year design storms), the precipitation infiltrates into the underlying outwash sands and gravels and is routed through the overburden groundwater aquifer to discharge in the creek. A primary objective of the design and construction of the stormwater management system is to maintain these characteristics.

The final design for the stormwater management system was developed through consultation with the City, the Conservation Authority and the Ministry of Natural Resources. The Ministry of Environment and Energy reviewed and issued the Certificates of Approval to construct the facilities.

The design consisted of the following components:

Lot Level Controls: Runoff from the roof and rear yards is directed over grassed surfaces to a swale/infiltration trench system. The swale/infiltration trench system is designed to route, collect and infiltrate the runoff from all events up to the 5 year design storm.

Conveyance Controls: The runoff from the driveway and road surfaces is routed through oil/grit manholes to pre-treat the runoff prior to the release to the end-of-pipe system. It was decided that direct infiltration of the road runoff was undesirable.

End-of-Pipe Controls: The infiltration basin/trench system implemented for the end-of-pipe control, through the centre of the site, has the capacity to collect, filter and recharge the runoff from all rainfall events up to the 100 year design storm for the entire development. The "first flush" (2 year) storm is infiltrated through the vegetated sand filter that extends over the bottom of the greenway for the full length of the facility. Runoff volumes greater than the "first flush" event up to the 100 year event are routed to an infiltration trench system constructed along both sides of the greenway basin.

An "Environmental Implementation Plan" was prepared to compile the information generated at the conceptual design stage plus the final stormwater management design into a single comprehensive document to guide construction. Edited excerpts from this document have been included focussing on the stormwater management design and the supporting documentation required for approvals **[Editor's Note: Appendices and non-essential figures have been omitted for purposes of conciseness]**.

The following drawings are attached in reduced format to show the construction details:

- A General Plan showing the layout of Phase I of the Subdivision (Figure I.10);
- Four Plan and Profile drawings showing the construction details for the end-of-pipe greenway system (Figures I.11, I.12, I.13 and I.14);
- One Plan showing the details of the rear lot infiltration gallery system (Figure I.15); and
- One Drawing showing the landscaping details for the end-of-pipe greenway system (Figure I.16).

I.4.1 Environmental Implementation Report

Excerpts from the City of Guelph's Environmental Implementation Report are outlined below, including the rationale, design criteria and analysis results used to design and construct the stormwater management system in the Pine Ridge Subdivision.

Introduction

The Pine Ridge Subdivision received Draft Plan Approval in July 1995. The conditions of Draft Plan Approval require that the following reports and/or plans be prepared to support the detailed final design of the subdivision and the implementation of the works:

i) Stormwater Management Report;
ii) Site Grading and Drainage Plan;
iii) Erosion and Sediment Control Plan; and
iv) Tree and Hedgerow Inventory and Conservation Plan.

Instead of separate documents, the above reports are combined into one comprehensive Environmental Implementation Report for the Pine Ridge Subdivision. The preparation of this document was guided by but supersedes all previous reports. The Environmental Implementation Report was jointly prepared by various consultants to specifically address Conditions 17 and 18 of the Ministry of Municipal Affairs and Housing and Conditions 19, 20, 27, 35, 37, 41, and 46 of the City of Guelph resolution dated June 12, 1995.

The Environmental Implementation Report is intended to govern and direct the design, construction, monitoring and maintenance of the services and stormwater management facilities in the Pine Ridge Subdivision.

Location

Figure I.7 shows the location of the proposed development and the surrounding area. The site is bounded by Gordon Street to the west, the Farley Farm to the south, by Ridgeway Avenue/Malvern Crescent to the north and by other lands owned by the developer in the annexed area east to Victoria Road.

Existing Conditions

a) Land Use

The existing land use for the Pine Ridge lands is agriculture. The predominant crops grown on these lands in recent years has been corn, beans and wheat. The adjacent lands to the east and south are also in agricultural production. The lands to the north (Malvern/Ridgeway) and west (Lowes/Dawn) have been developed with individual wells and septic systems.

b) Topography

The topography on the Pine Ridge lands is relatively flat. Most of the site slopes in a northwesterly direction toward Gordon Street. The north easterly part of the site slopes toward the kettle features in the Torrance Creek Watershed. The average gradient of these lands is 0.5%.

c) Soils

The predominant surface soil type throughout the Pine Ridge lands is Burford Loam (Wellington County Soils Maps). The hydrologic soil classification for Burford Loam is AB. The good drainage characteristics and high infiltration rates for this soil type have been verified by the geologic and hydrogeologic investigations completed for the Hanlon Creek Watershed Plan (Marshall Macklin Monaghan and LGL Ltd., 1993) and the Watershed Management Strategy for the Upper Hanlon Creek and its Tributaries (Gamsby and Mannerow Limited, Cumming Cockburn Ltd., and Code MacKinnon Ltd., 1993).

The kettle lake features located on the eastern boundary of the proposed development are identified as having a layer of muck soils in the bottom.

d) Water Table Monitoring

A network of water table observation wells was installed in the area of Clair Road and Gordon Street in 1988. Monitoring of the observation wells has been carried out on a bi-monthly basis since the installation. Figure I.8 shows the monitoring wells installed on the Pine Ridge property.

e) Site Constraints
The lands in the area of Gordon Street and Clair Road, and in this particular instance the Pine Ridge lands, do not have a typical or manmade drainage connection to Hanlon Creek or its tributaries. Therefore, surface runoff from the site is negligible. The combination of flat topography and permeable soils infiltrates the precipitation from the normal range of rainfall events. Only under extreme rainfall events such as a Regional Storm will surface runoff occur.

The recommendation of the Hanlon Creek Watershed Plan and the Watershed Management Strategy is that all runoff generated by the normal range of design storms (2 to 100 year events) be pre-treated and recharged to the shallow groundwater system.

Stormwater Management Criteria

The studies, policies and guidelines used to develop the stormwater management plan for the Pine Ridge Subdivision were as follows **[Editor's Note: some references may be out of date]**:

1) Hanlon Creek Watershed Plan, October 1993

2) A Watershed Management Strategy for the Upper Hanlon Creek and its Tributaries, June 1993

3) Environmental Impact Statement, Ariss Glen Developments, Torrance Creek/ Hamilton Corners, Class 2 Wetland Complex

4) Stormwater Management Practices Planning and Design Manual, 1994

5) Interim Stormwater Quality Control Guidelines, 1991

6) Stormwater Quality Best Management Practices, 1991

7) MTO Drainage Management Technical Guidelines, 1989

8) Urban Drainage Design Guidelines, 1987

Figure I.7: Location of the Proposed Development and Surrounding Area

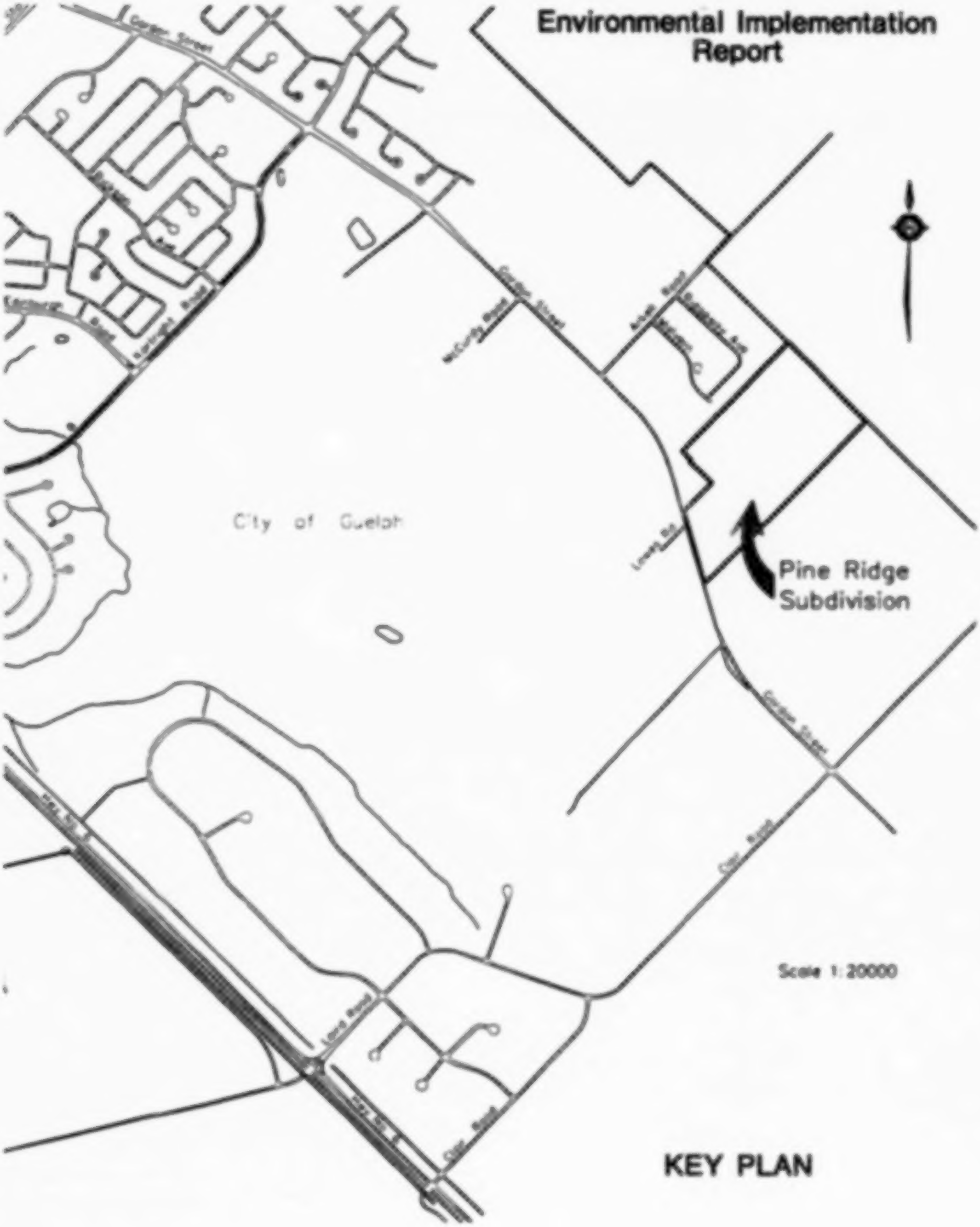
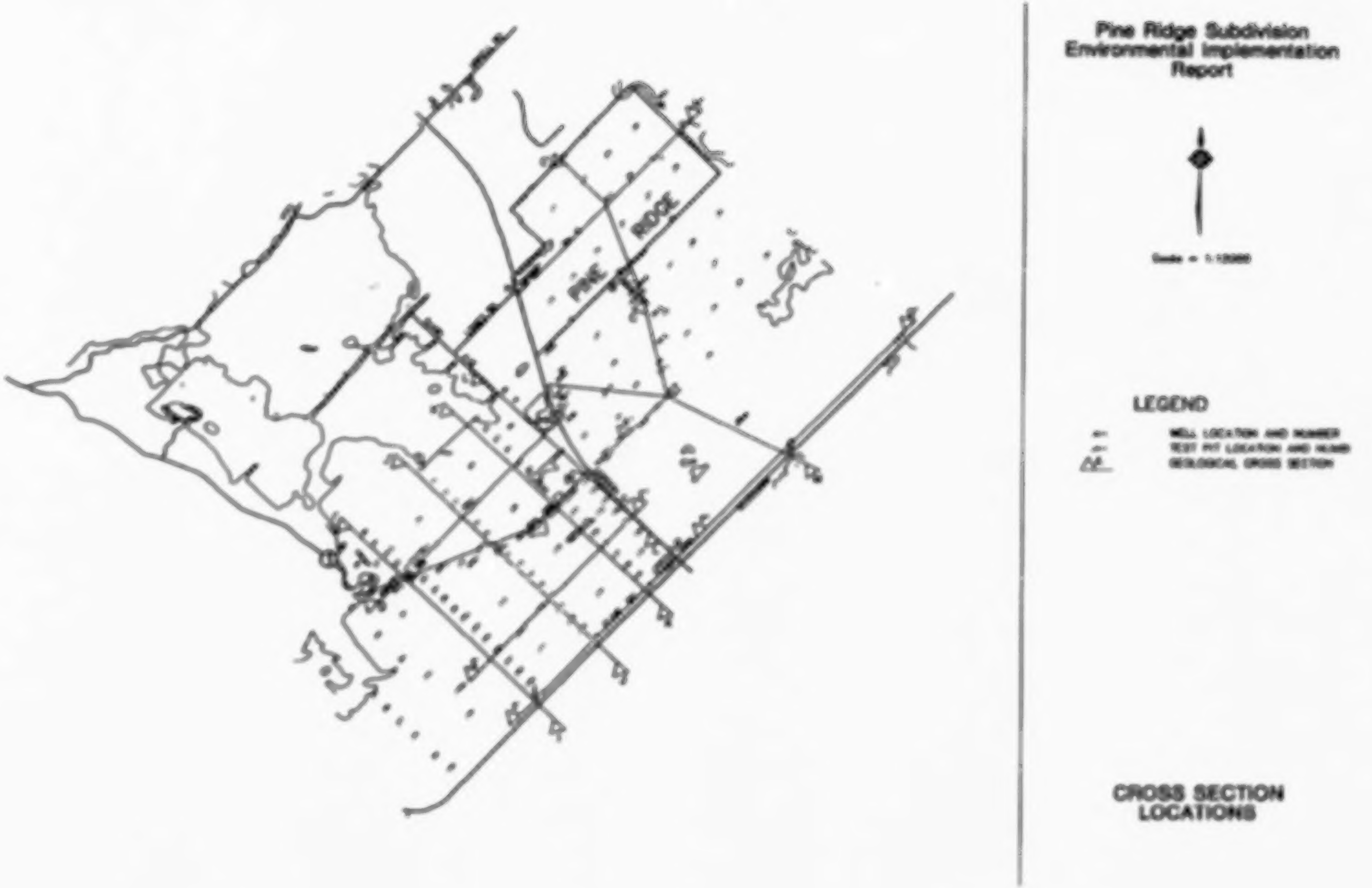

Environmental Implementation Report
Gordon Street
McCurdy Road
Arkell Road
Kortright Road
City of Guelph
Lowes Rd
Pine Ridge Subdivision
Clair Road
Laird Road
Hwy No. 6
Scale 1:20000
KEY PLAN

Figure I.8: Locations of Monitoring Wells on Pine Ridge Property


PINE
RIDGE
Pine Ridge Subdivision
Environmental Implementation
Report
Scale = 1:12000
LEGEND
WELL LOCATION AND NUMBER
TEST PIT LOCATION AND NUMBER
GEOLOGICAL CROSS SECTION
CROSS SECTION
LOCATIONS

The objectives of the stormwater management plan are as follows:

1) Promote the recharge of storm runoff on all grassed or pervious surfaces remote from the end-of-pipe system using lot level controls.

2) Promote runoff infiltration in the end-of-pipe system after suitable pre-treatment to remove sediments that could have a detrimental impact on the functioning of the greenway system.

3) Provide stormwater quantity controls for a range of design storms (up to the 100 year event). All events within this range of storms will be contained, treated and recharged in the greenway system. Adequate storage capacity will be provided to ensure that surface runoff will not occur under any rainfall event up to the 2 to 100 year design storms.

The method used to evaluate and design the stormwater management plan was as follows:

1) The mass rainfall data for the "first flush" design storm was generated using a two hour duration rainfall event. A three hour duration rainfall event was used to generate the mass rainfall data required to model the 5 and 100 year design storms. The Chicago parameters and the total depth of rainfall for each storm are as follows:

	First Flush	5 Year	100 Year
a =	743.000	1,593.000	4,688.000
b =	6.000	11.000	17.000
c =	0.799	0.879	0.962
r =	0.400	0.400	0.400
td =	120.000	180.000	180.000
Rainfall depth (mm) =	31.200	47.200	87.300

The Horton infiltration method was used in the runoff calculations. The infiltration parameters are very conservative to account for the long-term efficiency of the proposed drainage system. The parameters used in MIDUSS were as follows:

	Impervious Areas	Pervious Areas
Maximum Infiltration	0.0 mm/hr	76.0 mm/hr
Minimum Infiltration	0.0 mm/hr	13.0 mm/hr
Lag Constant	0.0 hr	0.25 mm/hr
Depression Storage	1.5 mm	5.00 mm

Based on the hydrogeologic investigation completed for the Watershed Management *Strategy for the Upper Hanlon Creek and its Tributaries*, the estimated soil permeability for the sand and gravel overburden found on the Pine Ridge site was 1.7×10^{-1} m^3/s. In the Hanlon Creek Watershed Plan, the estimated permeability for this soil type was found to range between 1 and 1×10^{-1} m^3/s. For design purposes, the coefficient of permeability for the soils on this site was reduced to 5×10^{-2} m^3/s. The more conservative permeability has been used to account for the expected long-term efficiency of the infiltration system.

The hydrologic model MIDUSS was used to create the runoff hydrographs and to route the flows through the storage and infiltration structures.

Stormwater Management Design Concept

Under the normal range of rainfall events, storm runoff does not occur from the Pine Ridge lands. The stormwater management system implemented for this development must emulate those existing conditions. To achieve this, the stormwater management system must have the capacity to infiltrate all runoff generated for the complete range of design storms up to and including the 100 year event. Only extreme events exceeding the 100 year storm are expected to generate surface flow that will leave the site.

The approach to stormwater management on this site must be an integrated one. This involves using the highly permeable soils throughout the development to start the infiltration process as close as possible to the point where the precipitation lands on the ground. The stormwater management system will include lot level, conveyance and end-of-pipe controls.

Lot level controls will include rear lot infiltration galleries **[Editor's Note: type of infiltration trench]**, flat swale drainage and sump pumps discharging to the rear yards. With minor exceptions, all roof drainage will be directed to the rear yard.

Multiple storm sewer outlets will direct runoff to the greenway to keep the size of the drainage catchments small and to minimize the length of the impervious flow paths. Pre-treatment will be provided by oil/grit manholes (to clean the runoff by removing sediments and oils).

The end-of-pipe system is the greenway that stretches through the center of the Pine Ridge Subdivision. The greenway is comprised of three terraces that will collect, clean, filter and infiltrate the runoff. Quality control will be achieved by routing the discharge from the storm sewer outlets through settling areas to further remove sediments. The runoff will then be distributed over the surface of the greenway for infiltration through the bottom of the greenway.

To provide stormwater quantity control, for events exceeding the quality control rates identified in the current policies and guidelines, inlet structures will convey the pre-treated runoff to a gallery for direct recharge to the permeable native soils.

The greenway system will be designed with the capacity to provide treatment and recharge for the entire development site under the full range of design storms (2 to 100 year events). The lot level controls will be enhancements to the greenway. The lot level facilities will reduce the volume of runoff being conveyed to the greenway, thereby creating reserve capacity within the greenway.

The description and function of the end-of-pipe stormwater management facilities are outlined below:

a) Greenway:

The greenway is continuous from Gordon Street at the southwest corner of the site to the kettle at the northeast corner,

i) to provide stormwater access as frequently and uniformly as possible throughout its length to minimize the runoff volume and velocity that will be discharged to the ground surface at any location.

ii) to distribute the infiltration function over as much of the site as possible to maximize pervious surface contact.

iii) to permit flows that will exceed the storage capacity of the greenway (greater than the 100 year design storm) to discharge directly to the Gordon Street right-of-way and the kettles.

The operational features of the greenway will be graded essentially flat from end to end,

i) to provide multiple access points for storm runoff and to minimize stormwater travel distance and time from any point in the subdivision. This will direct the rainfall from hard surfaces onto vegetated surfaces as quickly as possible.

ii) to distribute the runoff to the largest possible ground surface area to maximize the contact surface and thereby minimize the length of time necessary for infiltration to occur. This also utilizes the maximum surface area for aerobic bacterial activity, thereby maximizing the treatment of any biodegradable contaminants that may be carried to the greenway.

iii) to minimize the flow velocity within the greenway and thereby minimize the potential for erosion.

iv) perimeter of the site, thereby minimizing the potential for drainage problems. Even so, the new homes adjacent to Ridgeway and Malvern will be higher than the existing ones. This is due to the grading of the road to permit sanitary and storm servicing and to ensure the operation of the rear lot infiltration gallery.

v) to maintain the natural surface and ground water drainage divides.

vi) micro variations in the topography of the bottom of the greenway terraces, below the operational levels, will be incorporated to create a meandering undulating landscape to provide enhanced visual, ecological and habitat diversity.

The greenway has been designed with 3:1 side slopes,

i) to create a small terrace approximately a metre in height, from the rear lot line to the bottom of the greenway. This will maximize the flat area available for recharge.

ii) to create a protected area for the installation of the recharge galleries which will accept flows exceeding the infiltration capability of the greenway bottom.

iii) to create a flat area for maintenance access within the greenway and to define the edge of the public property.

iv) a variety of side slopes (3:1 to 6:1) will be incorporated throughout the greenway system to create an aesthetically pleasing landscape during final design.

The vegetation in the greenway has been ecologically selected,

i) to maintain the porosity of the soils and maximize the infiltration capability.

ii) to minimize the level of municipal maintenance required in the facility.

iii) to maximize the nutrient uptake from the runoff that reaches the greenway.

iv) to create an atmosphere of aesthetic, cultural, social and recreational interaction.

A 300 mm thick sand filter layer will be constructed for the entire length and width of the greenway bottom, at the interface between the native gravel soils and the topsoil that will be placed over top, to prevent fine surface soils and sediments from penetrating the gravelly native soils and interfering with natural percolation capacity.

At storm sewer outlets, a peat layer may be added to further filter and polish the runoff prior to spreading over the greenway bottom for infiltration.

The sand layer will also permit the use of the greenway for some stormwater management and sediment control during construction. Sand contaminated by sediment accumulations can simply be removed and replaced before the final landscaping.

The inlet distribution/gallery system will be constructed of perforated polyethylene field tile, fed by catchbasin inlet structures strategically placed along the length of the tile and the greenway.

The inlet distribution/gallery system will be located in the side slope terraces of the greenway, adjacent to the rear lot lines:

i) for more cover and protection than would be afforded in the middle of the greenway (protection from frost and damage by human activity).

ii) for aesthetic, inconspicuous placement of inlet structures.

The inlet elevation to the distribution/gallery network will be set at the 2 year storm flood storage level to ensure that all storms with runoff volumes less than the two year storm are infiltrated through the grassed surface of the greenway.

The bottom of the distribution/gallery will be located approximately 1.0 metre above the average high water table to ensure adequate soil contact to facilitate recharge.

b) Park:

The park block has soil capabilities similar to the greenway and can infiltrate not only its own rainfall, but also the runoff from adjacent lands. The park will receive runoff from the rear of Lots 7 to 12, 33 to 36, and the Malvern Crescent lots that slope toward them. Any runoff from the rear of Lots 37 to 42 will drain to the east, to Terrace 3000.

An infiltration structure is proposed along the rear of Lots 7 to 12, 33 to 36, and 37 to 42. A similar structure could be extended along Street B to ensure the collection and infiltration of any runoff that may be generated in the park.

The grading in the park has been prepared based on discussions with the City of Guelph's Parks and Recreation Department.

c) Quality Control:

Quality control on this site will consist primarily of keeping any sediment accumulations on the surface of the greenways. During construction, there could be accumulations of sediment requiring periodic clean up, especially before final landscaping.

After servicing and house building is complete, accumulations of sediment will be very small, course grained and will be distributed over a very large surface area. It will be many decades, if ever, before it will be necessary to clean up post-development deposits.

Stormwater Management Plan

The SWMPs in the Stormwater Management Practices Planning and Design Manual (1994) were screened and nearly all were found to be applicable to this site. Not all were selected, however, because not all will be acceptable to the municipality and not all are cost-effective.

A significant factor in the selection of SWMPs for this site is the "closed" nature of the stormwater management facility. There will be no runoff from the site until a rainfall exceeding the 100 year storm occurs. Thus, no routine rainfalls, sediments, or contaminants will discharge from the site to any other property or water body.

Sediment accumulation during construction is the only contamination of any concern, and this can be effectively managed through physical means, as described in Section 5.0(c) and Section 8.0 **[Editor's Note: these sections have not been included]**.

The selected SWMPs can be categorized as lot level controls, pre-treatment controls, and end-of-pipe controls (although the end-of-pipe controls could be considered "conveyance controls").

Lot Level Controls

Stormwater management practices recommended to provide lot level control on this site are as follows:

Flat Rear Yard Swales/Rear Lot Infiltration Galleries

Drawing 16 **[Editor's Note: drawing has not been included]** shows the profile of the rear lot line through Catchments 130, 160, 264, 287, 501 and 502 and the location and the construction details for the rear yard swale/infiltration gallery systems to be installed in Phase I. The grade on the rear yard swale is 1.0%. The lot grading can be adjusted to create a small amount of ponding at the rear of each lot (about 0.1 m). This will collect and infiltrate minor runoff from the rear yards. When rainfall events are intense enough to cause flow in the swale, the inlet structures will convey the runoff to the infiltration gallery for more direct recharge.

The analysis shows that the swale/infiltration gallery systems will collect and infiltrate the runoff from all events up to and including the 5 year design storm. Approximately 70% of the storage/discharge capacity of the infiltration gallery would be used to control the rear lot drainage for a 5 year event. The following tables summarize the runoff rate and volume and the routing results through the infiltration galleries in Catchments 130, 160, 264, 287, 501 and 502.

Table I.4: Uncontrolled Flow Rate and Runoff Volume

Design Storm	Catchment 130		Catchment 160		Catchment 264	
	Flow Rate m^3/s	Runoff Volume m^3	Flow Rate m^3/s	Runoff Volume m^3	Flow Rate m^3/s	Runoff Volume m^3
First Flush	0.024	32	0.068	94	0.053	72
5 Year	0.032	58	0.096	169	0.073	130
100 Year	0.064	135	0.177	392	0.142	300
Design Storm	Catchment 287		Catchment 501		Catchment 502	
	Flow Rate m^3/s	Runoff Volume m^3	Flow Rate m^3/s	Runoff Volume m^3	Flow Rate m^3/s	Runoff Volume m^3
First Flush	0.085	117	0.025	31	0.007	29
5 Year	0.118	211	0.035	55	0.080	214
100 Year	0.227	487	0.082	128	0.402	890

Table I.5: Stage/Storage/Discharge Capacities Comparison – Rear Yard Infiltration – Catchment 130 (Typical)*

Control Point	Available Capacity			Actual Capacity Used		
	Peak Flow m^3/s	Storage Volume m^3	Storage Depth m	Peak Flow m^3/s	Storage Volume m^3	Storage Depth m
Bottom of Gallery	0	0	0.00			
First Flush	—	—	—	0.02	6	0.21
5 Year	—	—	—	0.03	8	0.30
100 Year	—	—	—	0.05	18	0.66
Top of Gallery	0.08	27	1.00			

---

* *Editor's Note: The five tables for Catchments 160, 264, 287, 501 and 502 have been omitted for purposes of conciseness.*

Conveyance Controls

Figure I.9 shows the drainage sub-catchments, drainage areas and the location of the storm outlet for each catchment. The catchments are as small as possible to minimize the discharge at any one location. The multiple storm sewer outlets will reduce the potential sediment accumulation.

The direct infiltration of runoff is not acceptable because adequate pre-treatment cannot be provided. However, the conveyance methods that will remove sediment and other potential contaminants from the runoff prior to discharging to the greenway will be implemented.

It is recommended that oil/grit manholes be installed to pre-treat the runoff from the streets in Phase I prior to discharging it to the greenway.

Oil/grit manholes are recommended for catchments with a drainage area less than or equal to 2 hectares. The sub-catchments for this development meet that criteria.

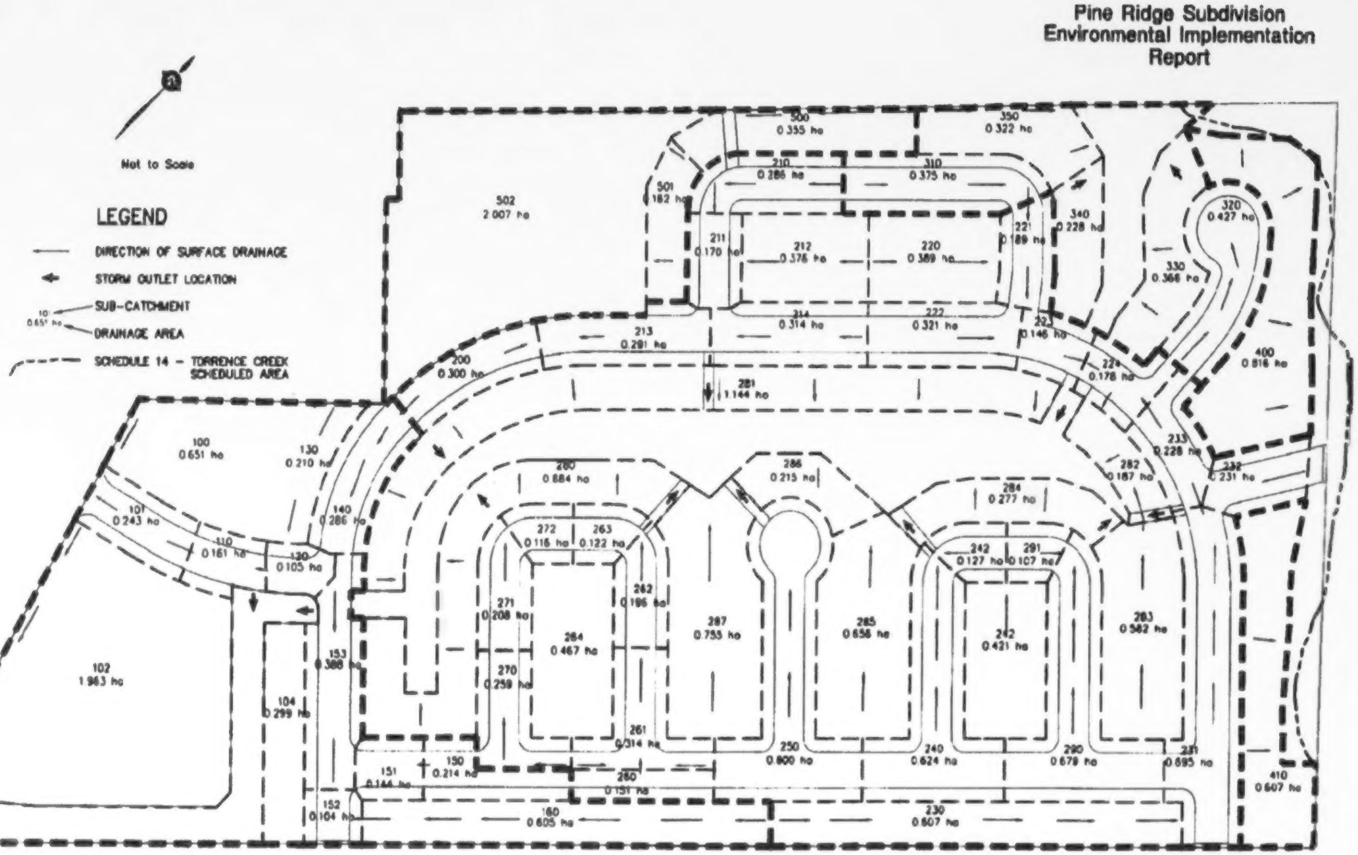
The removal of sediments through the use oil/grit manholes followed by the sediment forebays at the stormsewer outlets will provide pre-treatment of the runoff entering the greenway terraces.

End-of-Pipe Management

The end-of-pipe system is the greenway that stretches from Gordon Street through the center of the site to the kettle in the northeast corner. The greenway is comprised of three ponding terraces. The terrace at Gordon Street (Terrace 1000) is set slightly lower than the central terrace (Terrace 2000) which will be slightly lower than the kettle terrace (Terrace 3000).

The entire greenway system will be pre-graded as part of Phase I of the development. Terrace 1000 and part of 2000 will be constructed as part of Phase I. The rest of Terrace 2000 and Terrace 3000 will be topsoiled and seeded after pre-grading with the final construction occurring during the subsequent phases of the Pine Ridge Subdivision.

Figure I.9: Drainage Area Plan


Pine Ridge Subdivision
Environmental Implementation
Report
Not to Scale
LEGEND
DIRECTION OF SURFACE DRAINAGE
STORM OUTLET LOCATION
10
0.651 ha
SUB-CATCHMENT
DRAINAGE AREA
SCHEDULE 14 - TORRENCE CREEK SCHEDULED AREA
502
2.007 ha
500
0.355 ha
350
0.322 ha
501
0.182 ha
210
0.286 ha
310
0.375 ha
211
0.170 ha
212
0.376 ha
220
0.389 ha
221
0.189 ha
340
0.228 ha
320
0.427 ha
330
0.366 ha
400
0.816 ha
213
0.291 ha
214
0.314 ha
222
0.321 ha
223
0.146 ha
224
0.178 ha
200
0.300 ha
281
1.144 ha
233
0.228 ha
232
0.231 ha
100
0.651 ha
130
0.210 ha
280
0.884 ha
286
0.215 ha
284
0.277 ha
282
0.187 ha
101
0.243 ha
140
0.286 ha
272
0.116 ha
263
0.122 ha
242
0.127 ha
291
0.107 ha
110
0.161 ha
120
0.105 ha
262
0.196 ha
271
0.208 ha
264
0.467 ha
287
0.755 ha
285
0.656 ha
242
0.421 ha
283
0.582 ha
102
1.963 ha
153
0.386 ha
270
0.259 ha
104
0.299 ha
261
0.314 ha
250
0.800 ha
240
0.624 ha
290
0.679 ha
231
0.695 ha
151
0.144 ha
150
0.214 ha
260
0.151 ha
410
0.607 ha
152
0.104 ha
160
0.605 ha
230
0.607 ha

Pre-treated runoff from the street network is released to the greenway in small quantities at multiple locations. A settling forebay will be constructed at each of the storm sewer outlets. These areas will provide energy dissipation and some sediment removal. The attached drawings show the construction details for the greenway system. A planting scheme for the greenway system is detailed on the attached drawings and in Section 9.0 **[Editor's Note: these drawings and this section has not been included]**.

The discharge from the settling bays will spread over the adjacent surface of each greenway terrace and will infiltrate through the vegetated surface of the greenway. Any suspended solids will settle on the surface. A sand filter will be constructed in the bottom of the greenway to prevent the surface soils from clogging the permeable native soils below. The sand filter will allow plant roots to maintain a porous interface between the soil surface and the underlying permeable soils.

Runoff volumes exceeding the quality control depth (first flush/2 year design storm) will flow by way of a series of inlet structures into the distribution/gallery network for direct recharge to the groundwater system.

The attached drawings show the location and extent of the greenway gallery network in Phase I of the Pine Ridge Subdivision **[Editor's Note: see Figures I.10 to I.16 at the end of this appendix]**.

Summer Operation

The greenway is comprised of three terraces identified as Terrace 1000 (Gordon Street), Terrace 2000 (centre) and Terrace 3000 (kettle). The depth of the terraces from the pond bottom to top-of-bank (rear lot line) is 1.80, 1.20 and 1.20 metres, respectively. The active storage depth in each terrace is 0.95, 0.65 and 0.53 metres, respectively. The remaining depth in each terrace up to the overflow weir is freeboard. The greenway system has been designed to contain the 100 year design storms within the above storage depths. The freeboard provides reserve storage and attenuation capability for events exceeding the 100 year design storm while protecting basements and foundations.

Table I.6 lists the total flow rate and the total runoff volume received by the each terrace under the "first flush," 5 year and 100 year design storms.

Table I.6: Uncontrolled Flow Rate and Runoff Volume (Total) – Summer Operation

Design Storm	Terrace 1000		Terrace 2000		Terrace 3000	
	Flow Rate m³/s	Runoff Volume m³	Flow Rate m³/s	Runoff Volume m³	Flow Rate m³/s	Runoff Volume m³
First Flush	0.74	1,021	1.61	2,305	0.2	287
5 Year	1.02	1,738	2.32	4,066	0.29	509
100 Year	1.95	3,737	4.72	9,110	0.61	1,140

Table I.7 compares the greenway routing results with the available stage-storage-discharge capacities for each terrace.

Table I.7: Terrace 1000 – Stage/Storage/Discharge Comparison – Summer Operation (Typical)*

Control Point	Available Capacity			Actual Capacity Used		
	Peak Flow m³/s	Storage Volume m³	Storage Elev. m	Peak Flow m³/s	Storage Volume m³	Storage Elev. m
SURFACE CONTROL						
Pond Bottom	0	0	332			
First Flush	0.03	1,080	332.50	0.03	856	332.42
5 Year	—	—	—	0.1	1,304	332.57
100 Year	—	—	—	0.26	2,651	332.95
Weir	0.41	6,232	333.6			
Rear Lot Line	3.67	7,531	333.80			
GALLERY CONTROL						
Bottom of Gallery	0	0	332			
First Flush	—	—	—	—	—	—
5 Year	—	—	—	0.07	22	332.23
100 Year	—	—	—	0.23	77	332.79
Top of Gallery	0.29	97	333			

* *Editor's Note: The tables for Greenway Terrace 2000 and Terrace 3000 have been omitted for purposes of conciseness.*

The attached drawings indicate the storage elevations for the "first flush," 5 year and 100 year design storms for Phase I.

The estimated "drawdown" time for the "first flush" storm is 8 hours, for the 5 year storm 10 hours and for the 100 year storm 11 hours.

Water Quality

'Reasonable Use' Guideline MOE

The MOE 'Reasonable Use' guideline (Guideline B-7: The Incorporation of the Reasonable Use Concept into Groundwater Management Activities) was applied to this site to protect water quality.

Landscape Strategy and Vegetation

The following section presents the landscaping strategy for the Greenway System. The planting strategy has been designed to address concerns regarding the pre-treatment of stormwater runoff prior to infiltration to the groundwater system and the maintenance of the infiltration characteristics of the site.

The end-of-pipe control for the proposed development is provided by the Greenway System. Pre-treated runoff from the road network will be released to the Greenway at several locations. At each of these outlet points, a shallow (i.e., 30-45 cm deep) sediment forebay will be constructed to provide energy dissipation and additional removal of sediments, nutrients and heavy metals from the runoff.

Sediment Forebay Planting Strategy

The sediment forebays will be graded with 0.3 m of lower permeability soils (i.e., organics) to retain stormwater for a longer duration providing opportunities for the creation of a wet-mesic meadow habitat. By providing a wet meadow habitat within the sediment forebays, water quality can be enhanced through a variety of processes such as:

- sedimentation and physical filtration through root entrapment and sediment stabilization;
- adsorption to wetland vegetation, substrates and organic detritus; and
- nutrient uptake by plant root systems.

The wet meadow concept provides increased ecological, habitat and visual diversity. It is a safe method of pre-treating runoff since no permanent standing water will be present.

The following species that are tolerant of periodic short-term inundation with water have been recommended for planting within or around the margin of the wet meadow:

Typical Trees:

- eastern white cedar, tamarack, green ash, trembling aspen, balsam poplar, red maple, silver maple.

Typical Shrubs:

- red-osier dogwood, shrub willow spp., nannyberry, elderberry, chokecherry, Juneberry, grey dogwood, highbush cranberry, alternate-leaved dogwood.

The following low maintenance hydroseed mixture is recommended for ground cover establishment within the wet meadow:

25%	Canada blue joint grass (Calamagrostis canadensis)
25%	Rough-stalked meadow grass (Poa trivialis) or native substitute
20%	Highland Colonial bentgrass (Agrostis capillaris) or native substitute
15%	Creeping red fescue (Festuca rubra var. genuina)
5%	Tall White Aster (Aster lanceolatus)
10%	New England Aster (*Aster novae-angliae*)

The following native/non-invasive ground covers are recommended to supplement the above wet-mesic meadow seed mixture:

- sedge species, ostrich fern, virginia creeper, aster species, reed canary grass, tall manna grass, rattlesnake manna grass, common cattail.

Annual mowing with standard Parks and Recreation Department equipment following the release of plant seeds (i.e., fall) in the wet meadow/forebay areas is recommended to reduce biomass accumulation and maintain the infiltration characteristics of the site. Wetter areas within the sediment forebay area should be cut by hand rather than with a tractor to avoid disturbance to the vegetation and substrate. It should be noted that the annual detritus build up following mowing/dieback and subsequent decomposition is an important source of plant nutrients necessary to maintain the meadow system.

Balance of the Greenway

Once the storage volume within the settling bays is exceeded, runoff will flow in a diffuse manner into the adjoining greenway. A sand filter will be installed in the base of the greenway to trap sediments and maintain the permeability of the underlying native soils. The greenway will be vegetated with a low maintenance, successional dry-mesic meadow seed mixture to maintain and enhance the infiltration characteristics of the site.

Micro-grading of the greenway is recommended to create a gently undulating appearance. This will provide a slower and rougher conveyance system to promote additional sediment/pollutant removal and nutrient uptake by plants.

Micro-variations in the topography of the bottom of the greenway and a variety of side slope grades (i.e., 3:1 - 6:1) will also provide enhanced visual, ecological and habitat diversity. A 3 metre wide maintenance access with a permeable base (i.e., crushed limestone) is proposed around the perimeter of the greenway terraces and sediment forebays, and would be suited for a pedestrian walkway.

The following low maintenance successional dry-mesic meadow seed mixture is recommended for hydroseeding within the balance of the greenway:

25%	Canada blue grass (Poa compressa)
25%	Creeping red fescue (Festuca rubra var. genuina)
25%	Perennial ryegrass (Lolium perenne var. perenne)
10%	Red clover (Trifolium pratense)
10%	Black-eyed susan (Rudbeckia hirta)
5%	New England Aster (Aster *novae-angliae*)

Random clusters of native trees and shrubs are also recommended within the greenway to provide habitat diversity, aesthetic value and visual buffering. The following tree and shrub species are recommended for planting within the dry-mesic meadow component of the greenway:[*]

Trees:

- Eastern white cedar, white pine, white spruce, white ash, trembling aspen, balsam poplar, large tooth aspen, sugar maple, white oak and bur oak.

Shrubs:

- Elderberry, chokecherry, pin cherry, Juneberry, highbush cranberry, nannyberry, grey dogwood, red-osier dogwood and staghorn sumac.

Annual mowing with standard Parks and Recreation department equipment (as described above) is also recommended within this portion of the greenway to reduce the build up of detritus and maintain the infiltration characteristics of the site.[**]

[*] *This is not intended to be an all-inclusive species list. Other native/non-invasive species can be considered for upland side slopes and top-of-bank areas.*

[**] *It should be noted that 75% of the dry-mesic meadow seed mixture is comprised of the same grass species used in standard MTO applications. This grass mixture is generally low growing (i.e., 25 cm - 40 cm in height), requires little maintenance (once a year cutting), is drought/salt resistant and is also tolerant of periodic short-term inundation from runoff.*

Sediment and Erosion Control Plan

Prior to the start of construction activity, silt fencing will be installed along the property boundary. The silt fence will serve two purposes. The first will be to eliminate the opportunity for water borne sediments to be washed on to the adjacent properties. The second will be to delineate the environmental protection zones for trees and vegetation around the perimeter of the site. The ecologist will flag the location of the silt fence with the contractor to ensure that the drip line and root zones are protected.

Two types of silt fence will be used. Type 1 silt fence is a geotextile material attached to wooden stakes. Type 2 silt fence is steel T-bar fence posts with wire fencing to which a geotextile material will be attached. The Type 2 fencing will be placed in the more critical environmental preservation areas such as along the Torrance Creek/Hamilton Corners Wetland kettle formations and along the Norway Spruce hedgerow on the townhouse block (formerly commercial) in the northwest corner of the site near Gordon Street.

Once catchbasins or ditch inlets connecting to the infiltration galleries have been installed, the grates will be wrapped in filter cloth. This feature will be maintained until all building and landscaping has been completed in the individual drainage catchments.

A temporary berm will be placed at the upstream end of the first phase to prevent sediments from the following phases from contaminating the finished works. The procedure described above will be repeated for each phase of development.

Inspection and maintenance of all silt fencing and the temporary sediment pond will start after installation is complete. The fence and/or ponds will be inspected on a weekly basis or after a rainfall event of 13 mm or greater. Maintenance will be carried out, within 48 hours, on any part of the facility found to need repair.

Monthly reports on the condition of the sediment and erosion control measures will be submitted to the City of Guelph and the Grand River Conservation Authority.

Once all construction and landscaping has been substantially completed in a development phase, the sand filter will be inspected and any contaminated material removed. The sand filter will then be re-constructed and the final grading completed. The distribution/gallery system will then be installed and the terrace topsoiled and planted.

After construction of the complete development, erosion will not occur and sediment transport will be minimal. The swale drainage in the rear lot areas will be as flat as possible to minimize flow velocities. Sediment forebays at the storm sewer outlets will provide energy dissipation and sediment removal.

Maintenance Plan

A two-phase maintenance plan is recommended. Phase I will address the short-term, more intensive maintenance necessary during and immediately after construction. Once all landscaping has been completed, maintenance will shift to Phase II.

Phase I will include weekly inspections of all sediment control devices plus "as needed" inspection after any rainfall event exceeding 13 mm, with repairs completed within 48 hours of any damaged works and the collection of captured sediment. This work will be carried out by the consultant on behalf of the owner during the construction of the works. A monthly status report will be prepared and distributed to the City and the Conservation Authority.

Phase II will be the maintenance carried out by the City after all construction has been completed. This work will include the following:

i) The catchbasin sumps and the oil/grit type manholes will require pumping and cleaning twice a year (spring and fall) to remove accumulated silt.

ii) The sediment forebay areas will require a yearly visual inspection to determine sediment accumulation. When sediment removal is required, the surface of the forebay should be removed, the sand filter restored, and the recommended vegetation replanted.

iii) The remaining surface in the greenway should be mowed once a year. After many years, some areas of the terrace bottom may show signs of silt accumulation. If so, the surface should first be aerated. If this does not restore the infiltration characteristics, then the surface of the sand filter should be removed, re-constructed, topsoiled and reseeded with the recommended vegetation.

iv) The gallery system should be inspected regularly to ensure the system is draining. The inlets should be inspected seasonally to ensure that there is no blockage by leaves and debris.

v) The road grades throughout the development are flat (0.5%). The City of Guelph should re-evaluate their winter sanding practices to minimize the application within this development. This will reduce the potential impact from chlorides and sediments being directed to the greenway terrace system.

Conclusions

The stormwater management system has been designed to collect, clean, filter and recharge all the runoff up to the 100 year design storm within the boundaries of the development. Reserve capacity has been provided in the freeboard of the greenway terraces to store and attenuate more severe rainfall events such as the Regional Storm. The greenway system also creates an amenity for passive recreation by the residents.

From the foregoing analysis, the following conclusions are drawn:

1. The lot level controls will collect and infiltrate the runoff from roofs and rear lot drainage catchments, for all storms up to the 5 year event, through a rear lot swale/infiltration gallery system.

   This will reduce the volume of runoff directed to the greenway. It will also separate the cleaner roof and yard runoff from the potentially more contaminated runoff generated on the street right-of-way.

2. Oil/grit separators (pre-treatment controls) can pre-treat the road runoff prior to discharge to the greenway by removing sediments. This, in turn, will minimize any long-term deterioration of the infiltration function.

   The use of infiltration techniques on the municipal right-of-way is not recommended because of the limited ability to pre-treat the runoff.

3. The greenway system has been designed as a stand-alone system. The greenway terraces have the capacity to retain, filter and infiltrate the full range of design storms (up to the 100 year events) from the entire development. The "first flush" storm will infiltrate through the bottom of the greenway terraces. The larger design storms will be partially infiltrated and partially conveyed to the inlet distribution/gallery system for more rapid recharge.

   Under winter operating conditions, the greenway terrace system has the capacity to retain and infiltrate the runoff generated by a 100 year design storm.

4. The proposed stormwater management system for this development will maintain the existing surface and groundwater divides for all design storms up to and including the 100 year event.

5. Under Regional Storm conditions, the freeboard provided in the greenway terraces (approximately half the total storage depth) will provide further attenuation and storage prior to overflow to either Gordon Street in the west or the kettle to the east. This storage and attenuation will reduce the Regional Storm flows released from the site below pre-development levels.

   Any flows released from the site at Gordon Street will be directed to the north to match the existing drainage patterns. When development occurs to the south of this site, the City of Guelph and the Grand River Conservation Authority can consider re-directing the Regional Storm flows to the extended greenway system being proposed on those lands.

6. The major/minor system has the capacity to convey storm runoff to the greenway system under all rainfall events.

7. Development of this site will provide a net improvement to the water quality in the area by bringing sanitary services to existing residences and by reducing the amount of fertilizer and herbicides applied relative to the current agricultural practices.

8. Environmental management measures have been specifically developed to provide a high level of protection for vegetation and wildlife habitat features adjacent to the site. These measures include protective fencing, erosion and siltation control, infill plantings, naturalized buffer zones and a public education/awareness program.

9. The landscaping strategy for the Greenway System has been designed to address concerns regarding the pre-treatment of stormwater prior to infiltration to the groundwater system and the maintenance of the infiltration characteristics of the site. In addition to improving stormwater quality, the planting strategy has been developed to enhance habitat diversity and aesthetic value, and to provide passive recreational opportunities for the future residents of the proposed development.

In our opinion, the proposed stormwater management system meets the intent of the Hanlon Creek Watershed Plan and the Watershed Management Strategy for Hanlon Creek and its Tributaries.

References for Design Example 3 [Editor's Note: some references may be out of date]

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Figure I.10: General Plan showing the Layout of Phase I of the Subdivision

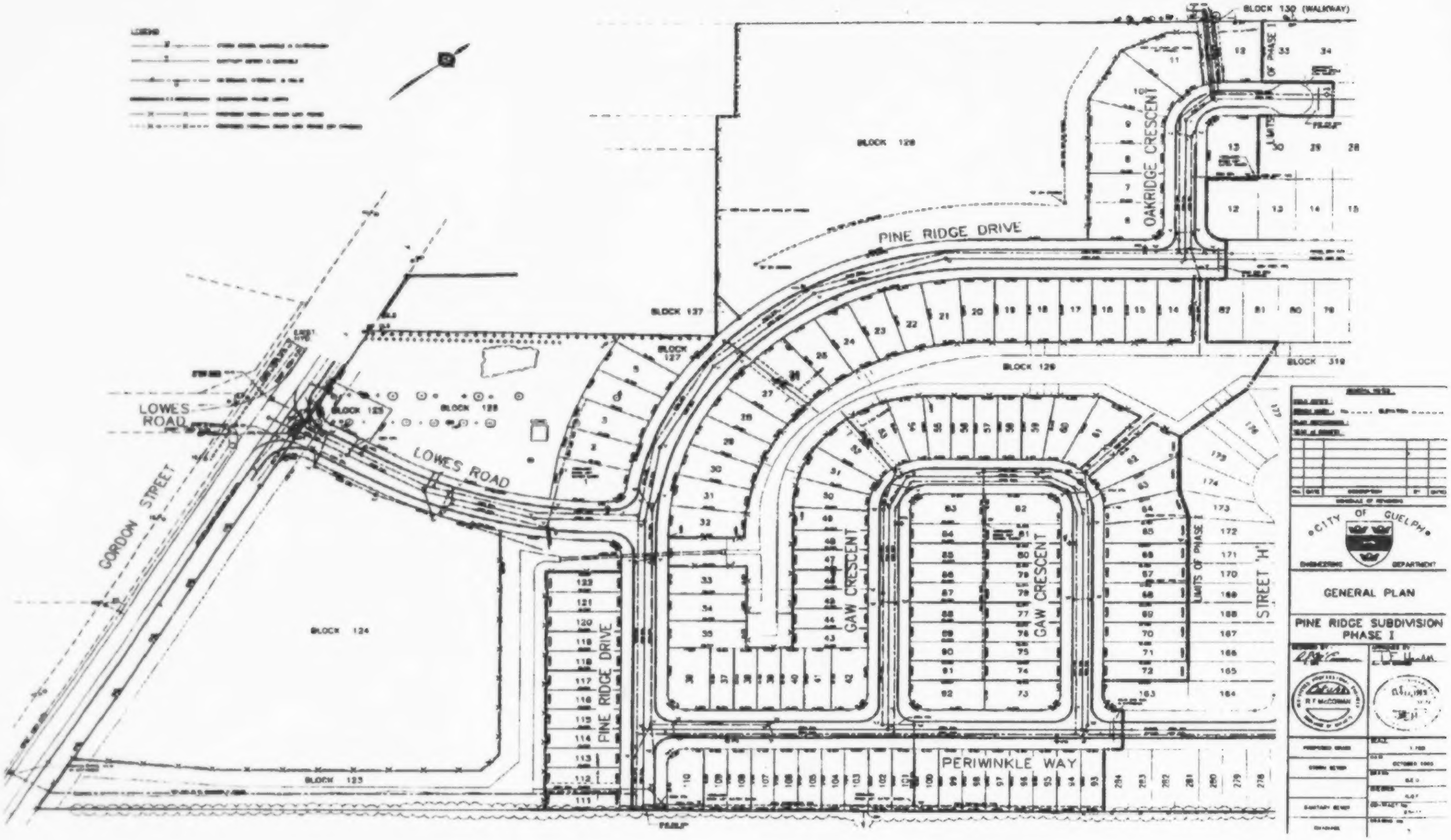

BLOCK 130 (WALKWAY)
LIMITS OF PHASE I
OAKRIDGE CRESCENT
BLOCK 128
PINE RIDGE DRIVE
BLOCK 137
BLOCK 127
BLOCK 129
BLOCK 319
LOWES ROAD
GORDON STREET
BLOCK 125
BLOCK 126
BLOCK 124
BLOCK 123
GAW CRESCENT
GAW CRESCENT
FINE RIDGE DRIVE
PERIWINKLE WAY
STREET 'H'
CITY OF GUELPH
ENGINEERING DEPARTMENT
GENERAL PLAN
PINE RIDGE SUBDIVISION
PHASE I

Figure I.11: Plan and Profile Drawing showing Construction Details for the End-of-pipe Greenway System

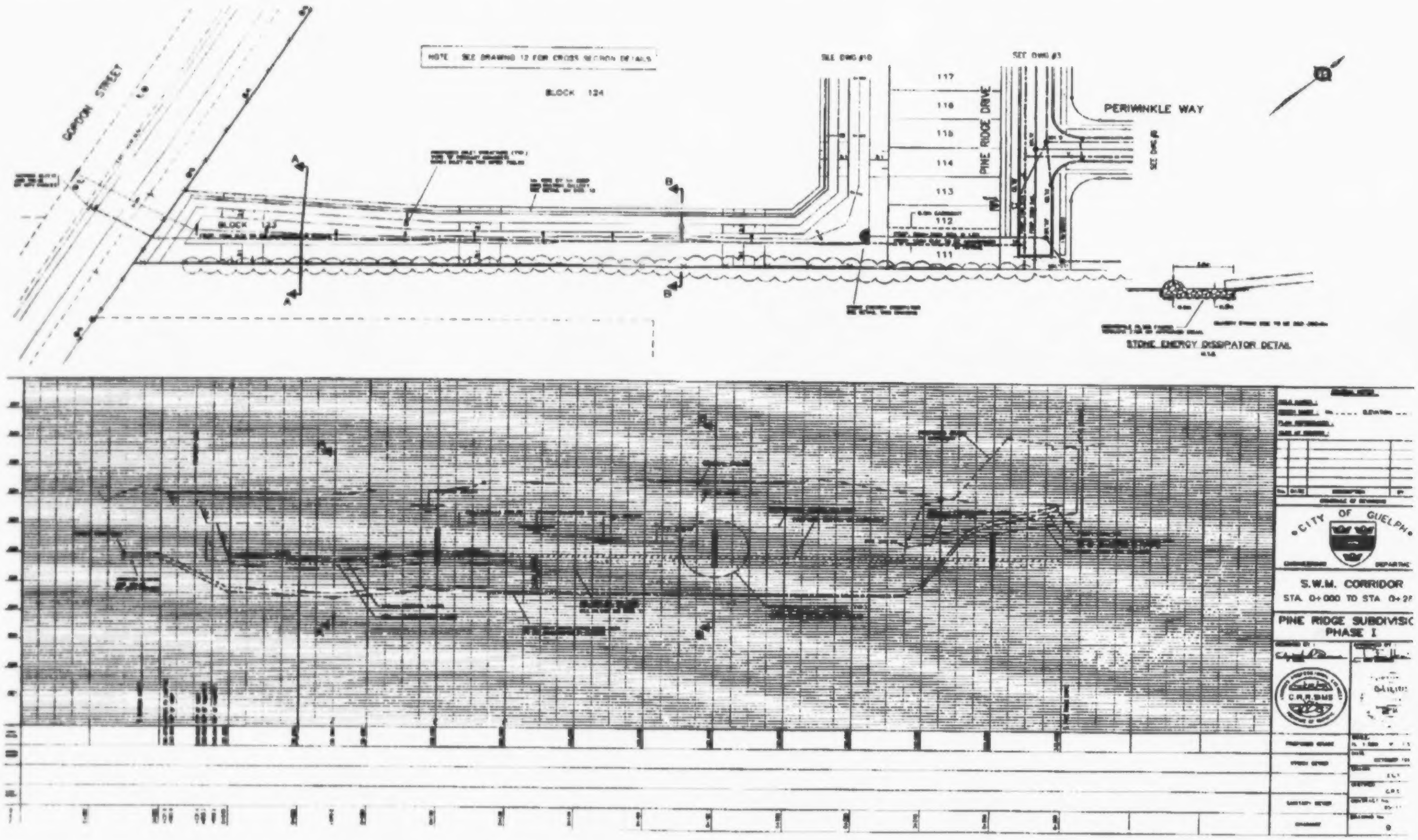

GORDON STREET
NOTE: SEE DRAWING 12 FOR CROSS SECTION DETAILS
BLOCK 124
BLOCK 125
SEE DWG #10
SEE DWG #3
117
116
115
114
113
112
111
PINE RIDGE DRIVE
PERIWINKLE WAY
A
B
STONE ENERGY DISSIPATOR DETAIL
CITY OF GUELPH
S.W.M. CORRIDOR
STA. 0+000 TO STA. 0+2
PINE RIDGE SUBDIVISIO
PHASE I
C.R.R.SMS

Figure I.12: Plan and Profile Drawing showing Construction Details for the End-of-pipe Greenway System

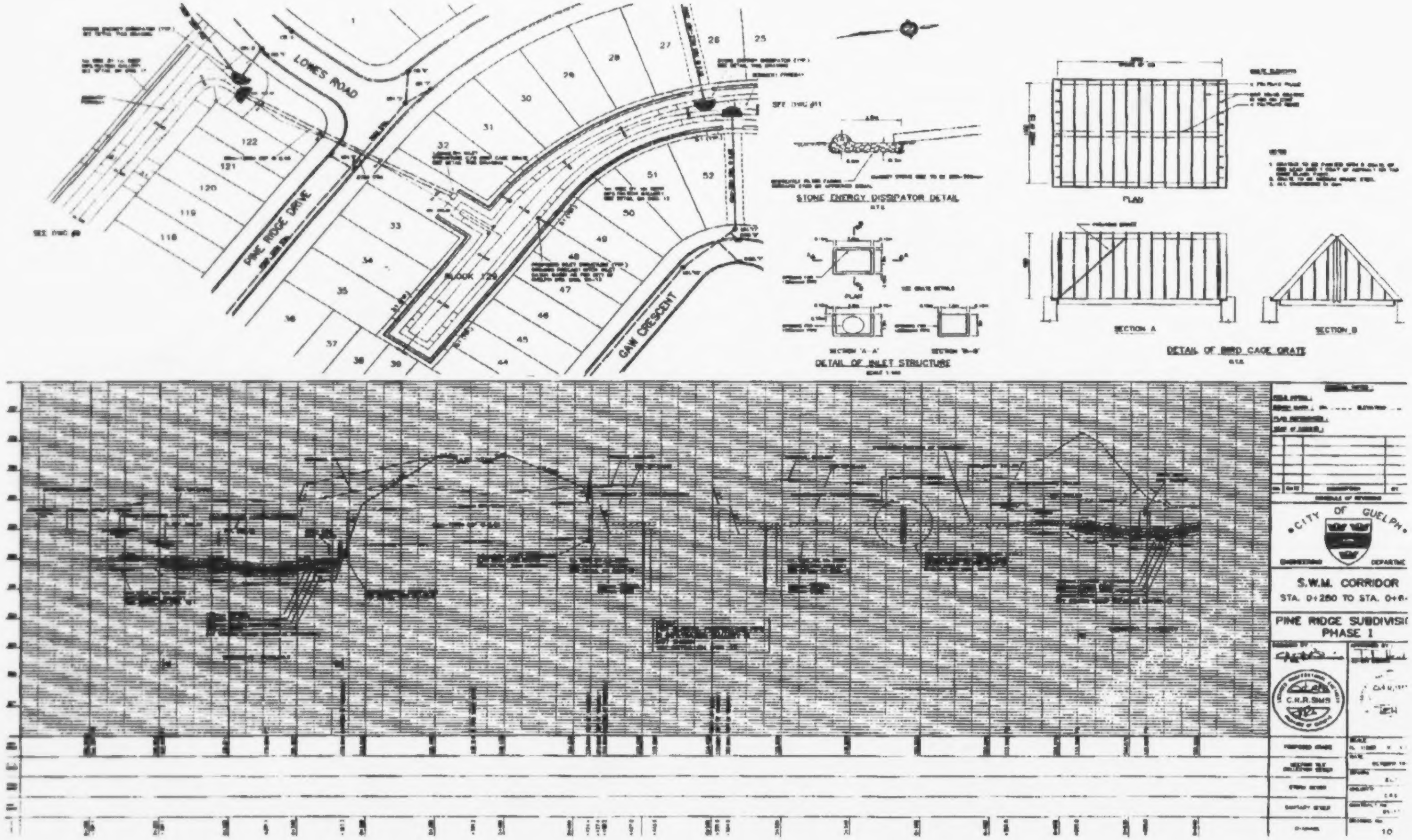

LOWES ROAD
PINE RIDGE DRIVE
GAW CRESCENT
BLOCK 129
1
25
26
27
28
29
30
31
32
33
34
35
36
37
38
39
44
45
46
47
48
49
50
51
52
118
119
120
121
122
SEE DWG #11
STONE ENERGY DISSIPATOR DETAIL
PLAN
SECTION 'A-A'
SECTION 'B-B'
DETAIL OF INLET STRUCTURE
PLAN
SECTION A
SECTION B
DETAIL OF BIRD CAGE GRATE
CITY OF GUELPH
S.W.M. CORRIDOR
STA. 0+280 TO STA. 0+6
PINE RIDGE SUBDIVISI
PHASE I
C.R.R. SIMS
10

Figure I.13: Plan and Profile Drawing showing the Construction Details for the End-of-pipe Greenway System

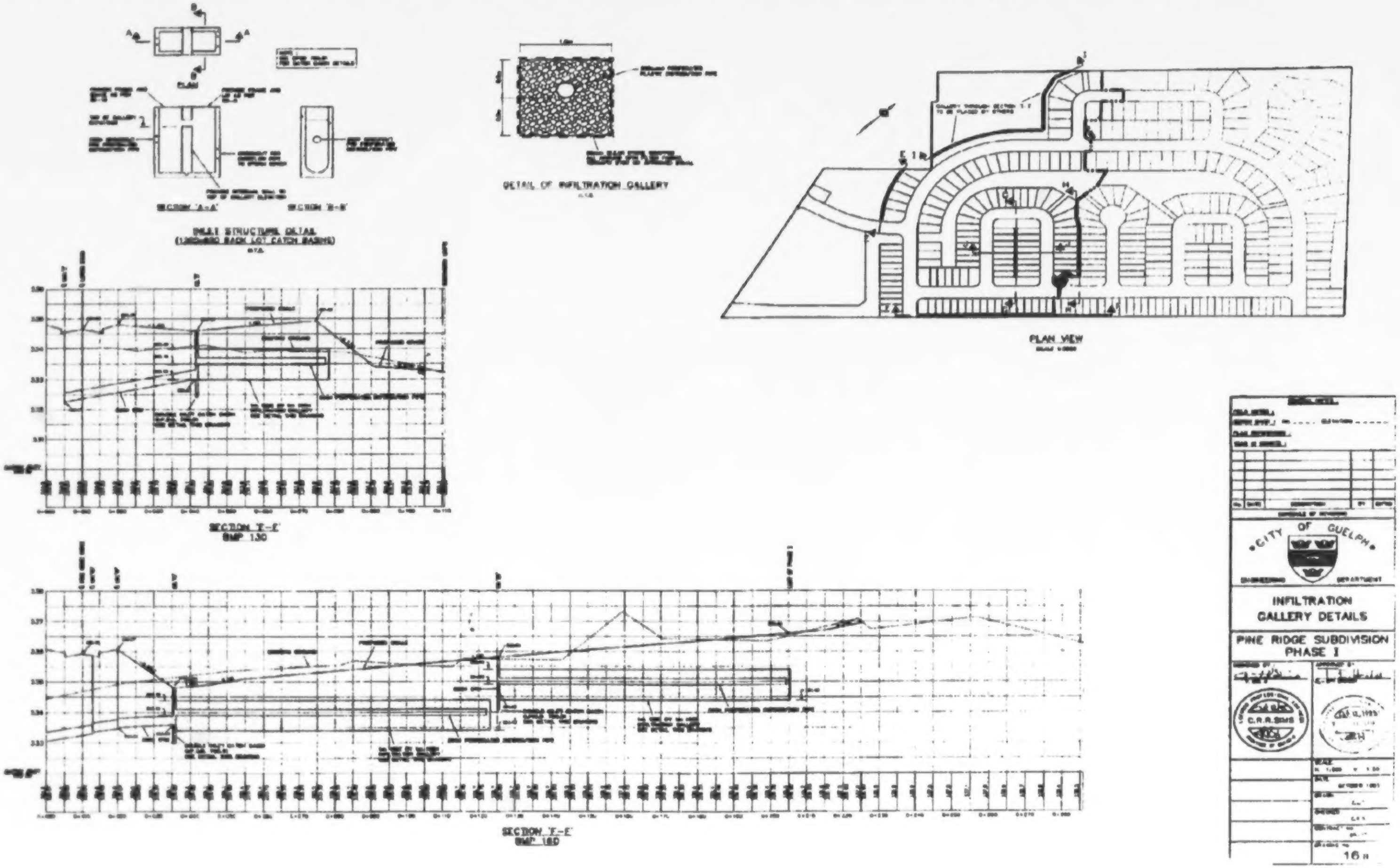

PLAN
SECTION 'A-A'
SECTION 'B-B'
INLET STRUCTURE DETAIL
(1200x600 BACK LOT CATCH BASINS)
DETAIL OF INFILTRATION GALLERY
PLAN VIEW
SECTION 'F-F'
BMP 130
SECTION 'E-E'
BMP 180
CITY OF GUELPH
ENGINEERING DEPARTMENT
INFILTRATION
GALLERY DETAILS
PINE RIDGE SUBDIVISION
PHASE I
C.R.R.SIMS
16

Figure I.14: Plan and Profile Drawing showing the Construction Details for the End-of-pipe Greenway System

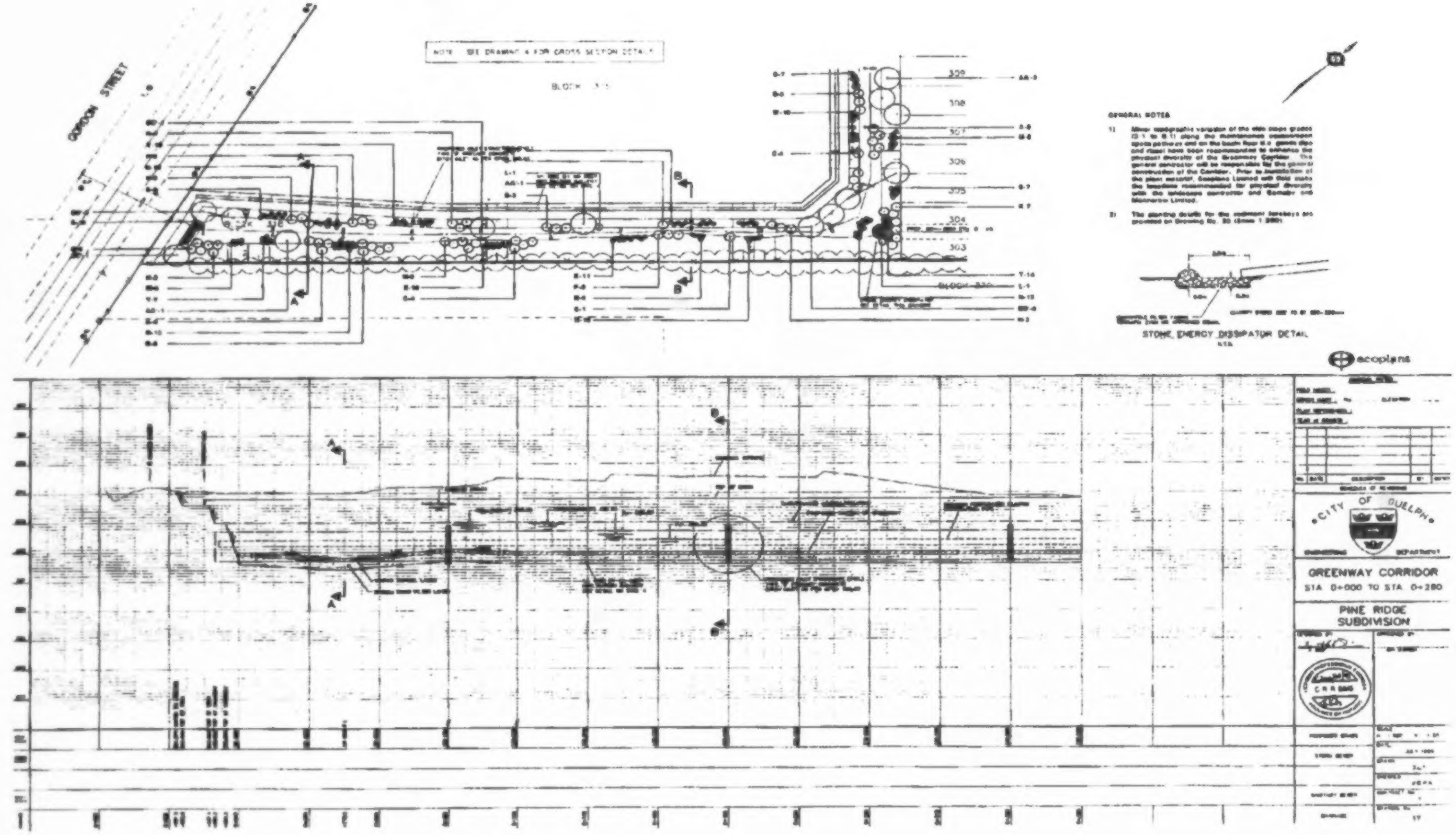
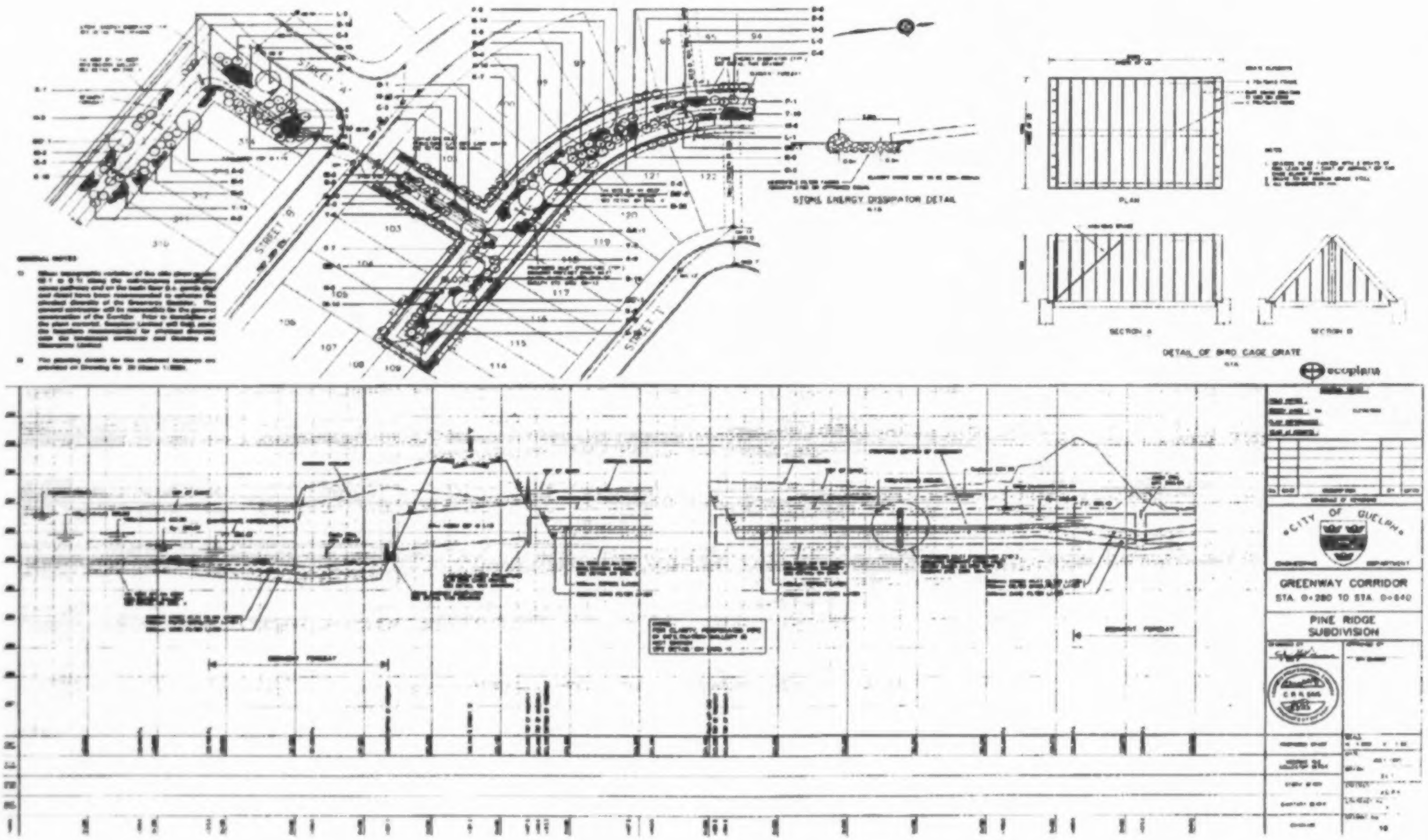

GORDON STREET
NOTE: SEE DRAWING 4 FOR CROSS SECTION DETAILS
309
308
307
306
305
304
303
GENERAL NOTES
STONE ENERGY DISSIPATOR DETAIL
N.T.S.
ecoplans
CITY OF GUELPH
ENGINEERING DEPARTMENT
GREENWAY CORRIDOR
STA 0+000 TO STA 0+280
PINE RIDGE SUBDIVISION

Figure I.15: Plan showing Details of the Rear Lot Infiltration Gallery System


GENERAL NOTES
STREET 8
STREET 7
310
311
103
104
105
106
107
108
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114
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116
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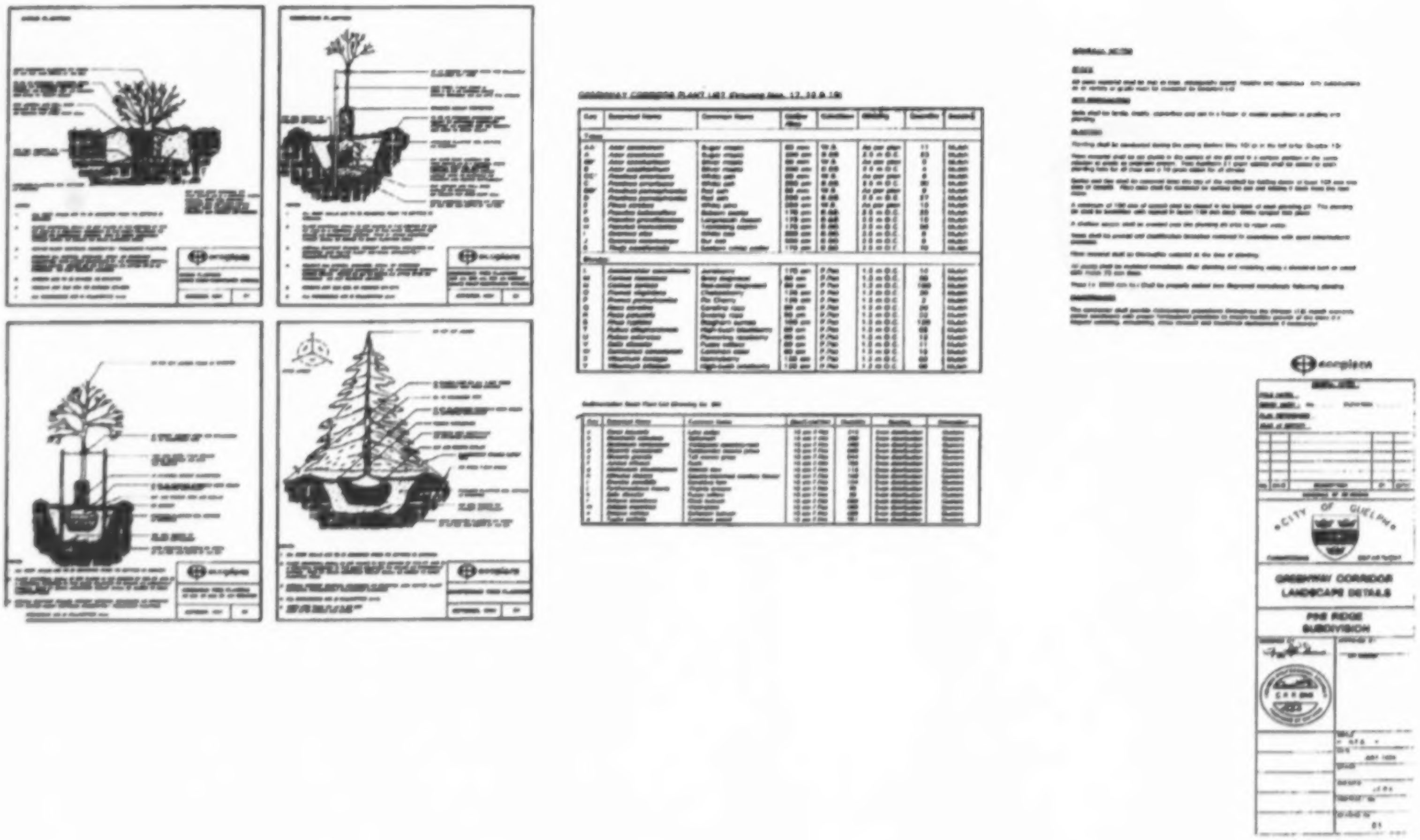
119
120
121
122
95
94
STONE ENERGY DISSIPATOR DETAIL
PLAN
SECTION A
SECTION B
DETAIL OF BIRD CAGE GRATE
ecoplans
CITY OF GUELPH
ENGINEERING DEPARTMENT
GREENWAY CORRIDOR
STA. 0+280 TO STA. 0+640
PINE RIDGE SUBDIVISION

Figure I.16: Landscaping Details for the End-of-pipe Greenway System


CITY OF GUELPH
GREENWAY CORRIDOR LANDSCAPE DETAILS
PINE RIDGE SUBDIVISION